WATER QUALITY CONSIDERATIONS IN THE DIRECT REUSE OF WASTEWATER AS A SOURCE OF POTABLE WATER SUPPLY

By

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CHAPTER I

INTRODUCTION

The direct reclamation or renovation of wastewater for the purpose of reuse can be looked at from two advantageous viewpoints, one of water quality management and one of water resources management. In some areas of the United States water quality deterioration due to wastewater discharges is the primary water problem, while in other areas water quantity is the crucial problem. Strict water quality standards are and will be adopted that will require treatment of wastewaters beyond conventional primary and secondary treatment, i.e., Advanced Wastewater Treatment (AWT), and the cost expended to treat the wastewater will make the water too valuable to be discharged into the stream, therefore it will be reused.

To see the need for water reuse from the aspect of water quantity one must look at the overall water supply and demand for the entire United States and for water short areas. Water demand in the United States for domestic and industrial use has risen from 40 billion gallons per day (BGD) in 1900 to 325 BGD in 1965 and is expected to be 900 BGD in 2000. To this the demand for irrigation water must be added which in 1965 was approximately 125 BGD and is

expected to be 180 BGD in 2000 (1). This represents a total demand of 450 BGD in 1965 and 1080 BGD in 2000. The average daily surface runoff is 1200 BGD of which only 550-650 BGD is economically available for use (1). This points to a water deficit of 400-500 BGD by 2000. In certain areas of the United States the water demand is already approaching the available supply.

The reuse of wastewater is considered as one of the primary methods for increasing the available water supply. The California Legislature (2), has affirmed that:

> "A substantial portion of future water requirements of the state may be met by reuse of reclaimed water, utilization of reclaimed water is in the best public interests, and that state should encourage development of reclamation facilities to help meet the growing water requirements."

The degree of purification required for direct water reuse will be determined by the specific reuse purpose that is intended for the water. Irrigation water for nonedible crops, parks, etc., is now being produced from conventionally treated wastewater that has been chlorinated. Reuse of wastewater for industrial purposes may require selective removal of corrosive or scale forming components. For recreational reuse, disinfection, removal of algae nutrients and elimination of aesthetically objectionable qualities such as foaming, odors, and color will be required. For reuse of watewater as a municipal water supply all of these components and characteristics plus many others must be removed before the water can be considered as a dependable,

safe, and aesthetically pleasing water supply.

This laboratory investigation was conducted to determine which chemical components and characteristics of wastewater are removed by the unit processes normally employed in conventional water treatment and those employed in advanced wastewater treatment. A further objective of the study was to determine which components or characteristics would "build up", i.e., increase in concentration through direct and continuous reuse of water.

CHAPTER II

LITERATURE REVIEW

A. Historical

Indirect reuse of wastewater is not new. For years man has discharged his wastewater into the stream only to be drawn out somewhere downstream to be used for municipal water supply. A 1962 survey by the United States Public Health Service (USPHS) found that 34-40 percent of the United States population derive their water supply from water that was, at least partially, wastewater at least once. The range of this reuse of wastewater was estimated to be from zero at high stream flows to 18 percent at low stream flows (1). Dry weather flows in the Thames River, which provides much of London's water supply, is estimated to be 33 percent wastewater effluent (2).

Planned reuse of wastewater for domestic purposes began in 1931 in California where a pilot plant was started in which secondary effluent was chemically precipitated, settled, filtered, and recharged to the ground water by surface spreading (5).

During the winter of 1939-1940 the Des Moines River in Iowa froze, and the city of Ottumwa, Iowa found itself drawing water from the river that was essentially diluted

raw sewage from Des Moines, Iowa 100 miles upstream. Des Moines was discharging 14 MGD of raw sewage into the river that had a stream flow at that time of 12 MGD. The assimilative capacity of the river, being frozen, was practically zero, and the river was merely acting as a closed conduit. The Biochemical Oxygen Demand (BOD) of the water at the entrance to Ottumwa's water treatment plant was 45 mg/l. Aesthetically pleasing water was impossible to produce and 106-116 mg/l of chlorine was required to provide "safe" water (5).

The most notable and most often quoted situation of water reuse for domestic purposes is that which occurred at Chanute, Kansas during October, 1956 to March, 1957. Due to a prolonged drought, Chanute's only water supply, the Neosho River went dry. An earthfill dam was constructed across the river just below the wastewater treatment plant's discharge and the wastewater effluent was allowed to back up to the water treatment plant's intake. This created essentially a stabilization pond with a theoretical detention time of 17 days. Through recirculation and post-chlorination the wastewater treatment plant was able to obtain an average of 86 percent BOD removal, 76 percent Chemical Oxygen Demand (COD) removal, 25 percent Alkyl Benzene Sulfonate (ABS) removal, and 67 percent removal of polyphosphates. The stabilization pond reduced the BOD another 75 percent, the total nitrogen and ammonia nitrogen by 55 percent and ABS by 50 percent.

Some qualitative and quantitative observations made during reuse were: the high ammonia concentration of the water made breakpoint chlorination impractical. Coagulation and settling was less effective. The sand filters became coated and plugged frequently, necessitating the removal of the sand for cleaning quite often. The treated water had several objectionable characteristics such as a pale yellow color, unpleasant musty taste and odor, and excessive frothing. The people generally blamed the water for any illness. The chloride concentration increased from 180 mg/l before reuse to 530 mg/l at the end of the reuse period. Noting an average incremental increase of 32 mg/l of chlorides, the water was estimated to have been reused 10 times. Other increases observed in the tap water were: sodium from 59 mg/l before reuse to 379 mg/l during reuse; total hardness, 108 mg/l to 180 mg/l; total alkalinity, 39 mg/l to 174 mg/l; nitrate-nitrogen, 1.9 mg/l to 2.7 mg/l; COD, 0 to 44 mg/l; and total solids, 305 mg/l to 1139 mg/l. Bacteriological and virological examinations were conducted regularly. Only one sample in 112 showed positive for coliforms in the tap water. No viruses were isolated from the tap water (14).

B. Health Aspects of Water Reuse

Northington et al., (15) states that the following questions must be answered before the reuse of wastewater for drinking water can be considered a "safe" practice:

"1) Will the public be protected against enteric and related infections resulting from the use of such water at all times?"
"2) Are the organics not removed by carbon adsorption injurious to health?"
"3) Is there a build up of any contaminants of public health importance through repeated recycling?"
"4) Since the drinking water standards are based on different concepts, how do we develop the needed information to write drinking water standards applicable to reused water?"

In reference to the chemical quality of water, Table I gives the incremental increase in the selected compounds or properties for one cycle of water use from tap to wastewater effluent. Data from three cities are presented (18,19).

Sodium which has been seen to build up rapidly presents a health problem. Although the USPHS Drinking Water Standards does not have a recommended limit for sodium it is known that some people have difficulties in ridding themselves of sodium (14).

Nitrate-nitrogen has a maximum allowable concentration of 45 mg/l according to the USPHS Drinking Water Standards. This concentration is based on the prevention of methemoglobinemia, an infant disease that causes a change in the infant's hemoglobin leading to suffocation. Although there does not appear to be a large increase in nitrate-nitrogen through one cycle of water usage (Table I) a paradox exists in that the better a wastewater is treated (secondary treatment) the higher the nitrate concentration (16).

Concerning the bacteriological and virological quality of reused water, there is disagreement as to whether the standard coliform test is a reliable indicator of water

TABLE I

INCREMENTAL INCREASE OF CONTAMINANTS IN ONE CYCLE OF WATER USAGE FROM TAP TO WASTEWATER EFFLUENT

Contaminant	City 1	City 2	City 3
	All Values in mg/l		
COD	143	133	-
Organic-N	2.2	-	-
NH_3-N	16	19	12
NO_2-N	0.3	-	-
NO_3-N	3.5	1	4
Alkalinity	122	96	85
Chloride	56	24	70
PO_4	24	25	20
SO_4	33	21	44
Calcium	23	-	7
Magnesium	7	-	40
Sodium	57	-	83
TDS	291	172	249

Source: T. H. Y. Tebbutt, "Sewage Effluents as a Source of Water", Effluent and Water Treatment Journal, V (1965), pp. 565-567, and 569, 573.

Source: R. L. Evans, "Addition of Common Ions From Domestic Use of Water", American Water Works Association Journal, LX (March 1968), pp. 315-320.

"safety". Because the coliform test does not analyze for, or give an indication of, the presence of enteric viruses, Mosely (22) states that the coliform test alone is not a reliable indicator of water safety. Still others, such as the American Water Works Association (AWWA) (25), maintain the reliability of the coliform test based on the fact that there are thousands of coliform organisms present for each

enteric virus in raw wastewater.

The fate of viruses in conventional and advanced wastewater treatment has been studied on both a laboratory scale and on a full scale. It was found that activated sludge treatment is effective in removing or inactivating 80-90 percent of the enteric viruses whereas trickling filter treatment is relatively ineffective. Chemical precipitation followed by sand filtration has shown to be effective in removing 98.5-99.8 percent of the enteric viruses from secondary effluent. Lime was found to be the most effective coagulant because it raises the pH (15,21). Activated carbon has shown to be successful in adsorbing viruses (24). The remainder of viral inactivation is left to chlorination. The effectiveness of chlorination on viruses is another area of question. Some viruses are easily inactivated by chlorine while others are very resistant (25).

C. Unit Processes

1. Conventional Wastewater Treatment

Conventional primary and secondary treatment of wastewater is, of course, a prerequiste for advanced wastewater treatment. Northington, et al., (15) has said that the most important thing about primary and secondary treatment is that water reuse for potable water supply is impossible unless a good quality secondary effluent is produced continuously.

2. Chemical Precipitation or Flotation

Chemical clarification is almost always the first process employed in AWT. Clarification is accomplished by either precipitation or flotation. The purpose of chemical clarification is three-fold: remove essentially all the suspended solids; remove the phosphorous and other like compounds; and condition the water for other processes. A coagulating chemical is used to accomplish the removal and conditioning. Common chemicals used are aluminum sulfate (alum), ferrous sulfate, ferric chloride, and lime (either quick lime CaO or hydrated lime $Ca(OH)_2$). The purpose of coagulation is to form floc particles that can be removed by either precipitation or flotation.

Van Vuuren, et al., (26) and Hannah (31) favor the use of lime as a coagulant because of economics, solids handling (lime can be regenerated from the sludge by recalcination in a furnace), and its bactericidal-viruscidal effect. Lime must be used if ammonia stripping is part of the AWT. In addition lime treatment does not contribute to increasing the salinity since its chemical reactions act in softening the water and causing a reduction in Total Dissolved Solids (TDS). Hannah (31) reports that the required lime dosage is dependent on the wastewater characteristics, principally alkalinity and hardness. The alkalinity present in the water affects the required lime dosage needed to raise the pH to the level where phosphorous compounds will become insoluble. For low alkaline wastewaters, i.e.,

less than 25 mg/l alkalinity as $CaCO_3$, a pH of 10 can be obtained with 50-75 mg/l of CaO. Wastewater with an alkalinity of 600 mg/l would require almost 400 mg/l of CaO to obtain a pH of 10.

Stander and Van Vuuren (42) and others (26,28,29,31) all report a reduction in COD and BOD of the secondary effluent by chemical precipitation or flotation. Part of this reduction is due to the removal of biological solids and the remainder is due to the removal of soluble organic matter. Reductions of 50-90 percent COD have been obtained.

3. Filtration

Filtration is utilized to remove the floc particles, colloidal particles, and color causing matter that carry over from chemical clarification. Total filter thickness is usually in the range of 24-48 inches. Hydraulic loading is generally 2-5 gpm/ft^2. When headloss through the filter becomes excessive, due to clogging, the filter can be backwashed with clear water.

Shireman (39) reports on a common type of filter used now in both AWT and conventional water treatment. This type of filter is referred to as a multi-media filter in that it has at least two types of media. One of the more common multi-media filters has a layer of anthracite coal, a layer of fine sand, a layer of garnet, and a support layer of gravel.

4. Ammonia Stripping

Farrell (33) and Shireman (39) have reported on the

theory and operation of ammonia stripping at South Lake Tahoe's AWT plant. In a secondary effluent the major portion of the total nitrogen concentration is usually in the form of ammonia-nitrogen. At a pH of less than 8.0 the ammonia is present as ammonium ion. If the ammonium ion is converted to dissolved ammonia by raising the pH to above 10.0 then the ammonia can be removed or stripped by contacting the water with air.

The water is broken up into droplets by trickling down through a slat filled tower usually 20-30 feet in height. The ammonia is stripped from the droplet by the force of the air. About 400 cubic feet of air is required per gallon of water.

Efficient operation is very much ambient air temperature dependent. Cold air will rapidly cool the water which will increase the solubility of the ammonia. If the air is below $0^{o}C$ freezing of the water at the air inlet will occur making the tower inoperable. Another problem is that of scale in the form of $CaCO_3$. The scale forms because the previously lime treated water is supersaturated with $CaCO_3$, plus CO_2 in the air will further supersaturate the water. This scaling can be excessive and completely plug the tower. If the scale does not adhere strongly to the slats it can be removed intermittently by a water jet. Keeping the pH as low as possible helps prevent scaling, but the removal efficiency suffers somewhat.

In spite of the problems associated with ammonia

stripping, greater than 90 percent removal has been obtained at South Lake Tahoe's AWT plant.

5. Activated Carbon Adsorption

Researchers at the Robert A. Taft Water Research Center (34-36) and the Federal Water Quality Administration (37) have reviewed the literature, investigated actual operations, and conducted lab studies on the use of activated carbon in wastewater treatment. From these findings it was determined that, at the present time, granular activated carbon is more advantageous than powdered activated carbon, and that the COD in a well treated secondary effluent can be reduced to less than 3 mg/l by granular activated carbon adsorption. The granular carbon is retained in packed columns and the water passes through the column in either a downflow or an upflow manner.

The adsorptive capacity of the carbon is usually in the range of 0.40-0.50 pounds of soluble COD per pound of carbon. The main operating parameter is the contact or residence time. The contact time is figured as the time required to fill the empty column. A contact time in the range of 50 minutes is recommended even though about 90 percent of the removal occurs within 10 minutes of contact. It was found that hydraulic loadings between 4-10 gpm/ft^2 do not have an effect on removal efficiency.

Economic utilization of granular activated carbon is only possible because the carbon can be regenerated and reused. There are losses in the removal capability on the

order of 5 percent when the carbon is regenerated.

6. Demineralization

Dryden (38) has reviewed the three principal methods of demineralization of water: reverse osmosis; electrodialysis; and ion exchange. He concluded that ion exchange is the most developed and most promising process at this time. Ion exchange can be made to demineralize water by using a cationic exchange resin, with H^+ as the exchange cation, followed by an anionic exchange resin, with OH^- as the exchange anion. The resins are retained in packed columns. When the exchange resin is exhausted it can be regenerated with a concentrated solution containing the initial ion.

An ion exchange unit operated at Pomona, California received water with an initial TDS concentration of 600 mg/l and was able to reduce this to 70 mg/l (38).

D. Contaminant Removal Capabilities for the Unit Processes

Stander and Van Vuuren (40) have prepared diagrams of what each unit process, previously described, has shown to remove in actual operation. The reduction of COD, total nitrogen, phosphorous, TDS, and bacteria-viruses are presented in the following figures.

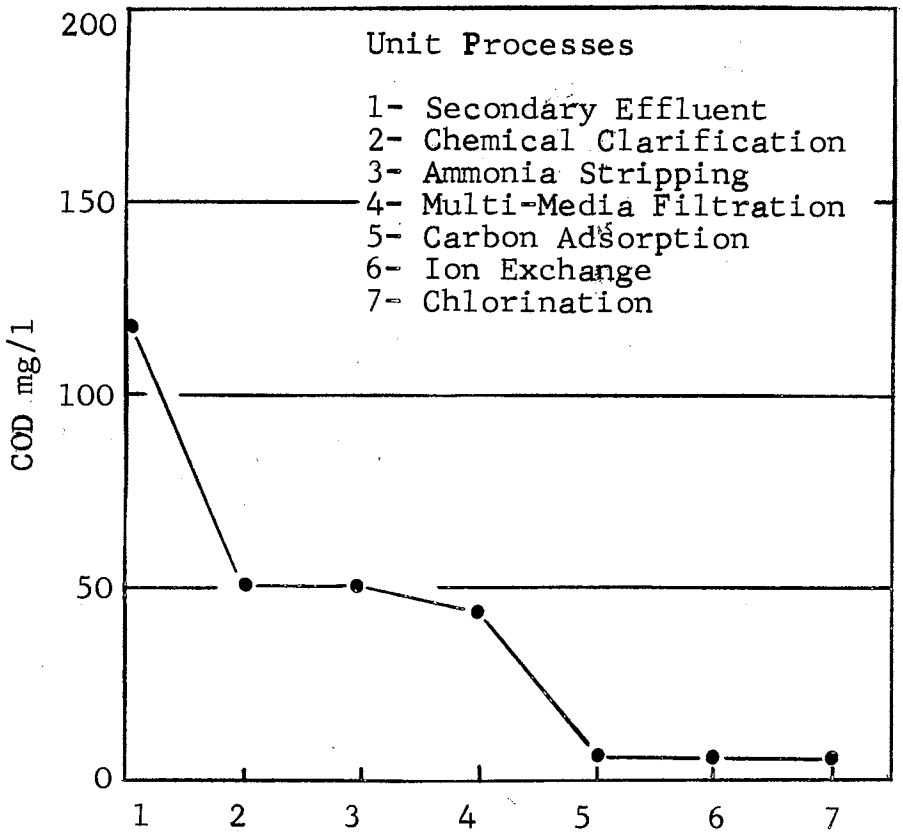

Figure 1. Reduction of Chemical Oxygen Demand. (After Stander and Van Vuuren 40)

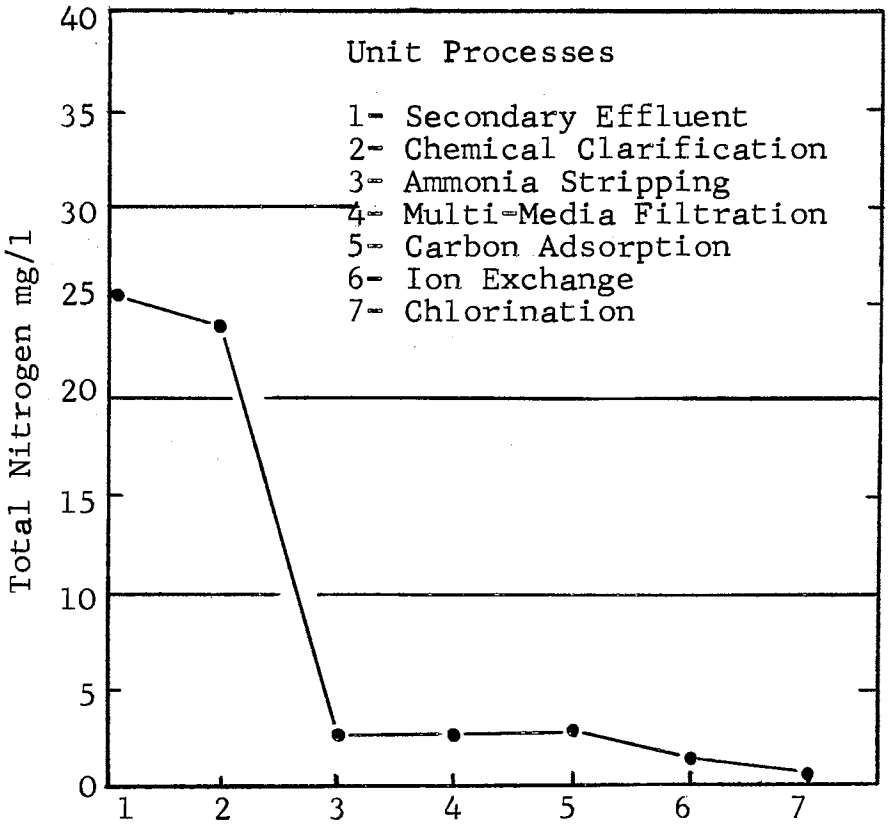

Figure 2. Reduction of Total Nitrogen. (After Stander and Van Vuuren 40)

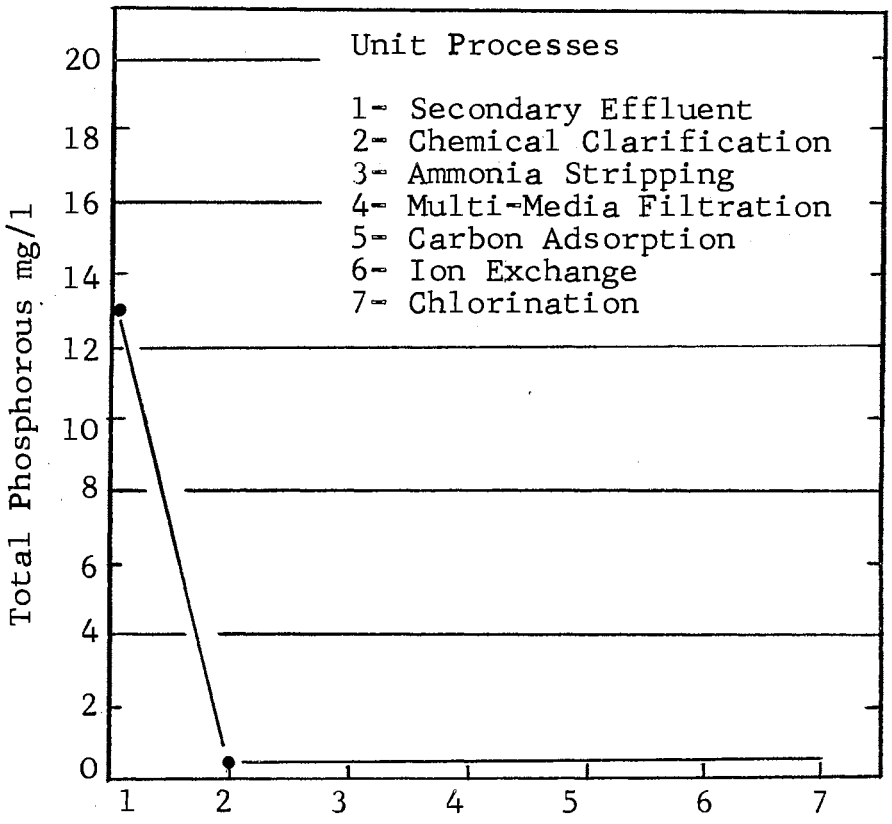

Figure 3. Reduction of Total Phosphorous. (After Stander and Van Vuuren 40)

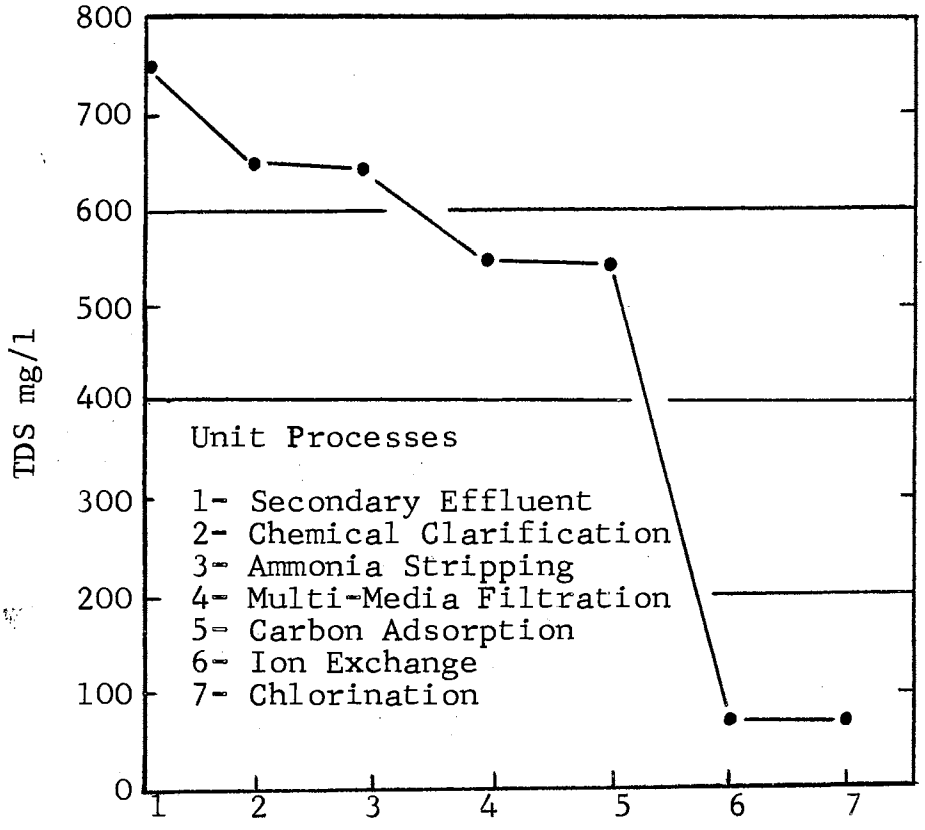

Figure 4. Reduction of Total Dissolved Solids. (After Stander and Van Vuuren 40)

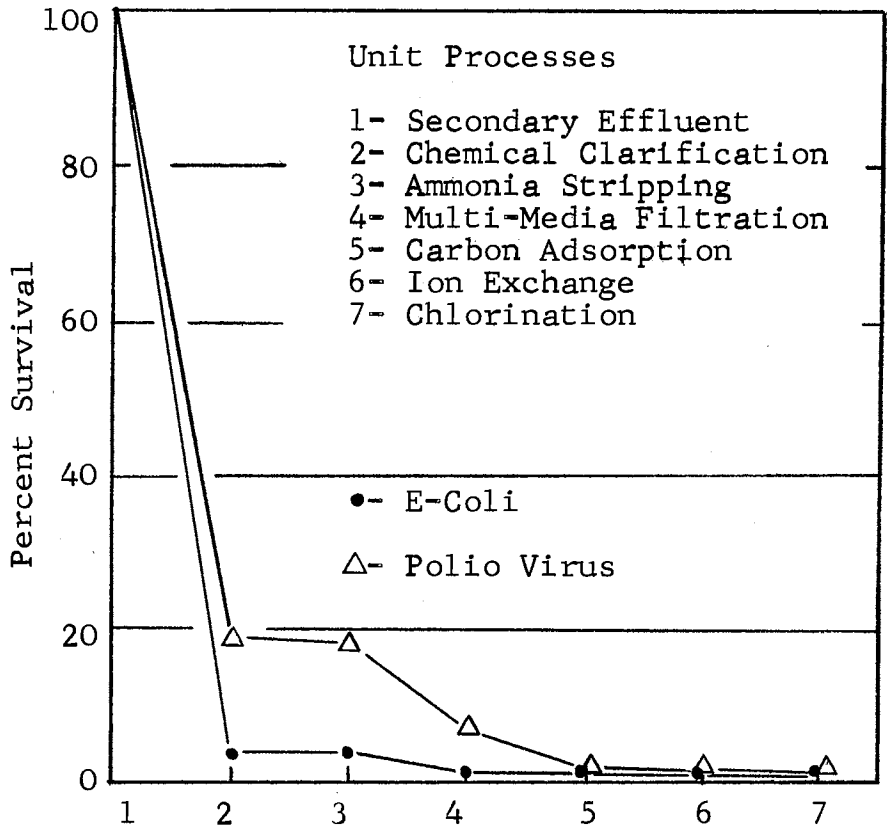

Figure 5. Removal and Inactivation of E-Coli and Polio Virus. (After Stander and Van Vuuren 40)

CHAPTER III

MATERIALS AND METHODS

A. General

To accomplish the objectives of this study, two phases of investigation were conducted. The first phase modeled conventional water treatment directly following conventional wastewater treatment. The second phase modeled advanced wastewater treatment directly following conventional wastewater treatment. A synthetic wastewater of known composition was utilized which permitted continuous reuse of the water. Selected chemical analyses were conducted to obtain the desired information about the removal capabilities of the unit processes, the possible build ups, and the quality of the water in general.

B. Synthetic Wastewater

A synthetic wastewater was made by adding four separate solutions to tap water for the initial cycle and afterwards adding the four solutions to the reused water for each subsequent cycle. The solutions used were a glucose solution to provide a carbon source, a salt solution to provide a nitrogen source and other necessary compounds, a phosphate buffer solution to provide a phosphorous source and buffering action, and a sodium chloride solution to

provide excess chloride for investigation purposes.

Addition of the solutions resulted in a wastewater with the following concentrations:

Constituent	Made Up At	Added At	Resultant Conc.
Glucose	50 gm/l	10 ml/l	500 mg/l
Theoretical COD = 534 mg/l			
$(NH_4)_2SO_4$	250 gm/l	1 ml/l	250 mg/l
Theoretical N Conc. = 54 mg/l			
$MgSO_4 \cdot 7H_2O$	100 gm/l	0.5 ml/l	50 mg/l
$MnSO_4 \cdot 7H_2O$	10 gm/l	0.5 ml/l	5 mg/l
$FeCl_3 \cdot 7H_2O$	0.5 gm/l	0.5 ml/l	0.25 mg/l
$CaCl_2$	7.5 gm/l	0.5 ml/l	3.75 mg/l
KH_2PO_4	26.35 gm/l	5 ml/l	131.7 mg/l
K_2HPO_4	53.50 gm/l	5 ml/l	267.5 mg/l
Theoretical P Conc. = 78 mg/l			
NaCl	10 gm/l	10 ml/l	100 mg/l

C. Systems and Processes

1. Start-Up

An activated sludge unit having a 10 liter volume was started by adding an initial seed of primary settled sewage from the muncipal sewage treatment plant at Stillwater, Oklahoma. The unit was batch fed once a day until a good flocculent system was established with a biological solids concentration in the range of 2000-2500 mg/l. There after it was attempted to maintain the solids concentration within this range by providing cell recycle. Sufficient air was continuously applied using 4 diffused air aerators to provide complete suspension of the cells and good mixing.

2. Continuous Flow Operation

Figure 6 is a flow diagram of the treatment process utilized in Phase One and Phase Two of the study. The synthetic wastewater was made up in 20 or 40 liter volumes

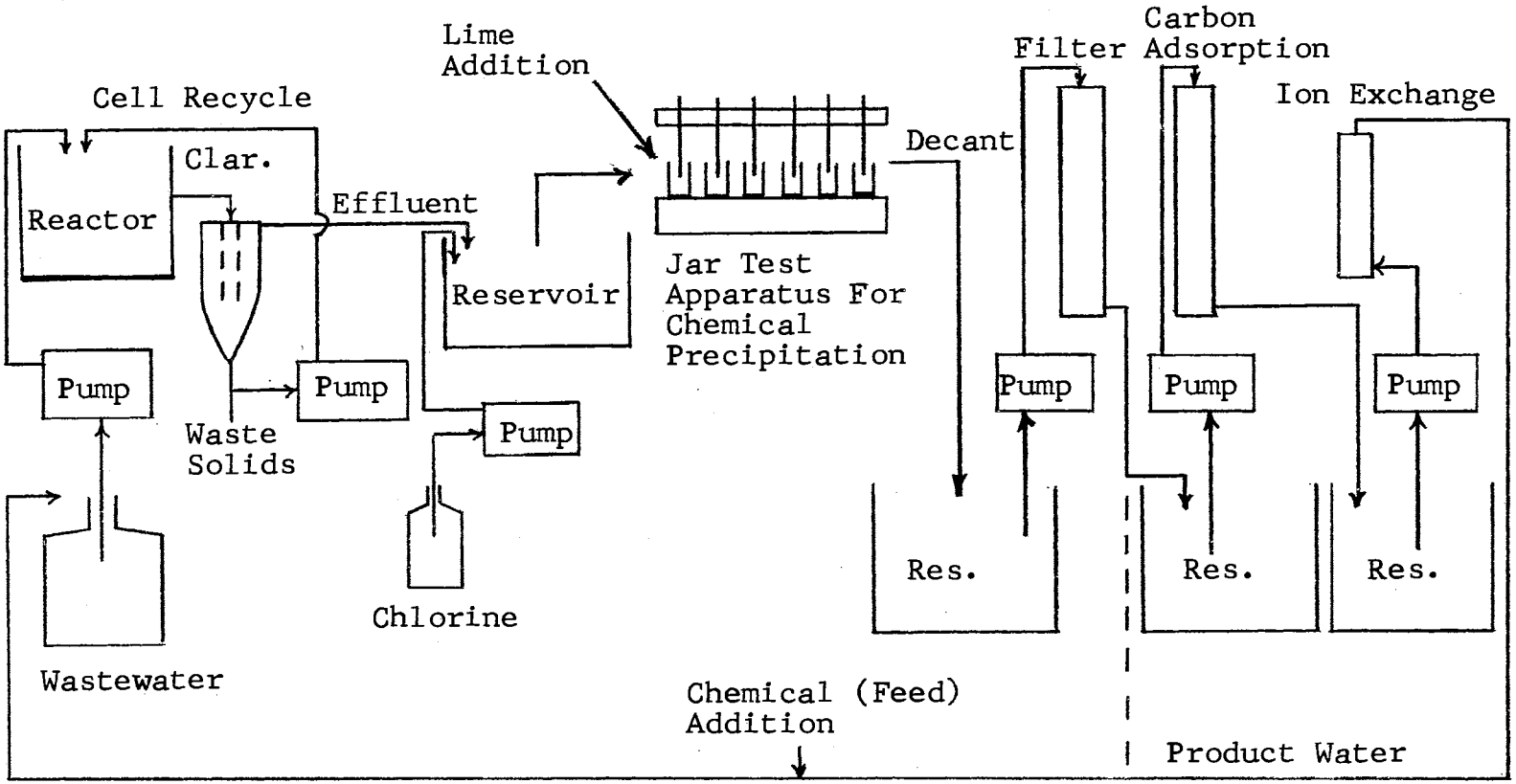

Figure 6. Flow Diagram of Treatment Process.

and pumped into one end of the activated sludge unit at a flow rate of 41.7 ml/min to provide a 4 hour theoretical detention time. This provided a COD loading of 0.0706 lb/day. The feed pump utilized was manufactured by Cole-Parmer Instrument and Equipment Company.

The effluent from the reactor discharged into a conical shaped clarifier with a volume of 5 liters. The effluent entered through a center well of 1 inch diameter and 10.5 inches of depth thus making the clarifier in essence an upflow clarifier. The effective diameter in the settling region of the clarifier was 5 inches, thus providing a horizontal surface area of 0.136 square feet. This gave an overflow rate of 0.081 gpm/ft^2. Cells were concentrated in the bottom of the clarifier and intermittently recycled back to the reactor, or wasted, to maintain a solids concentration of 2000-2500 mg/l in the reactor. A Sigmamotor Pump, model OV-22, was used to recycle the cells. The clarified effluent discharged from the clarifier via an overflow weir.

The clarified effluent entered a storage reservoir where it was chlorinated with 8 mg/l of available hypochlorate. To provide the chlorination, a solution of calcium hypochlorate was prepared such that 1 ml of solution was equivalent to 0.25 mg of hypochlorate. The solution was pumped at 1.35 ml/min by a Milton Roy Controlled Volume Mini Pump.

From the storage reservoir the effluent was manually

transferred to 1 or 2 liter beakers, lime was added to provide a concentration of 200 mg/l $Ca(OH)_2$ (see description of Phase One for further detail on lime addition), and a Phipps and Bird 6 paddle "Standard Jar Test Apparatus" was used for chemical precipitation. A quick-mix cycle of 30 seconds at 100 rpm's was provided, followed by a slow mix cycle of 20 minutes at 20-25 rpm's, and then 30-60 minutes of sedimentation. The liquid was decanted off and transferred into a storage reservoir. The precipitated sludge was disposed.

Filtration of the water was the next process employed. The water was pumped to the top of the multi-media filter by a Milton Roy Controlled Volume Pump, model R-220A, at a flow rate of 812 ml/min to provide a hydraulic loading of 3 gpm/ft2. The filter had an inside diameter of 3.625 inches and a total media depth of 27 inches, 3 inches of support gravel, 18 inches of fine sand with an effective size of 0.5 mm and a uniformity coefficient of 1.7, and 6 inches of anthracite coal with an effective size of 0.84 mm. The discharge from the filter was controlled by a valve such that a constant head of 6 inches was maintained. The filtered water discharged into a storage reservoir. The filter was backwashed with tap water after each cycle or more often when clogging was apparent. A bed expansion of approximately 50 percent was obtained during backwashing.

Activated carbon adsorption followed filtration. "Darco" granular activated carbon manufactured by Atlas

Chemical Industries Inc. was utilized. The carbon had an effective size of 0.84 mm (#20 mesh) and was retained in a packed column to give 36 inches of carbon depth plus 3 inches of gravel support. The water was pumped to the top of the column utilizing the same type of pump as that used for filtration. The hydraulic loading was 3 gpm/ft^2 and a constant head of 3 inches was maintained by regulating the discharge. This provided a contact time of 7.5 minutes based on the empty column volume. The water discharged into a storage reservoir.

The last process utilized was ion exchange demineralization. The unit was a Bantam Demineralizer, model BD-1, manufactured by Barnstead Still and Sterilizer Company. Mixed bed cartridges manufactured by Fisher Scientific Company provided the actual ion exchange resin. The cartridge had a recommended maximum throughput of 10 gal/hr; therefore a throughput of 6 gal/hr or 378.5 ml/min was selected. Flow through the cartridge was upflow at a hydraulic loading of 1.28 gpm/ft^2. The same type of pump as that utilized for filtration was used. A conductivity meter on the discharge side of the cartridge measuring ppm as NaCl indicated when the cartridge should be replaced. It was recommended that the cartridge should be replaced when the meter read 10 ppm as NaCl and this was adhered to throughout the investigation.

The product water from the demineralizer became the water that was used to make up the synthetic wastewater for

the next cycle. This permitted the operation of a closed system where the water could be reused continuously.

D. Phase One of Study

The study was separated into two phases, each having 5 cycles of water use. Therefore the water was reused 4 times during each phase.

Phase One of the study was intended to model a conventional water treatment system directly following conventional wastewater treatment. Therefore the treatment process utilized only included those unit processes up through multi-media filtration and did not include carbon adsorption or ion exchange. The water discharged from the filter thus became the water that was used in making up the wastewater for the next cycle. Operating the system in this manner would allow, among other things, the determination of which compounds and contaminants would build up with reuse of the water.

For cycle number one approximately 100 liters of water was processed, and it was this amount of water, minus unavoidable losses, that was reused for the remaining 4 cycles. Since it was desired to have a definite break between cycles, continuous flow to the activated sludge unit was stopped for a matter of hours, after all the wastewater for that cycle had gone through the reactor, until enough water could be processed through chemical precipitation and filtration to start the next cycle.

Since it was not known what coagulant to use or at

what concentration, a study was conducted using the clarified effluent to determine the best coagulant and dosage. It was decided to base this study on obtaining at least 90 percent removal of phosphorous. Alum and lime were selected as possible coagulants and the concentration added was varied between 0 and 200 mg/l. Alum did not yield good removal up to 110 mg/l when compared with lime, and therefore it was not considered for use. Figure 7 shows the results obtained, and from these results it was decided to utilize 200 mg/l of lime for all the remaining work.

Due to the lime addition the resultant pH of the product water was above 9.0, and it was felt necessary to neutralize the water before reuse. Therefore a titration curve was prepared using a solution of 1-Normal H_2SO_4. The water was neutralized to a pH of 7.6 throughout the remainder of Phase One. Refer to Figure 8 for the titration curve.

E. Phase Two of Study

Phase Two of the study included carbon adsorption and ion exchange in the treatment process. Like Phase One, the wastewater was made using tap water for cycle one and the reused water for the subsequent 4 cycles. Approximately 120 liters were processed for cycle one.

Cycles four and five differed from the normal treatment process in that a separate clarifier was not used for cycle four and the activated carbon column was not used for cycle five. For cycle four it was attempted to use the

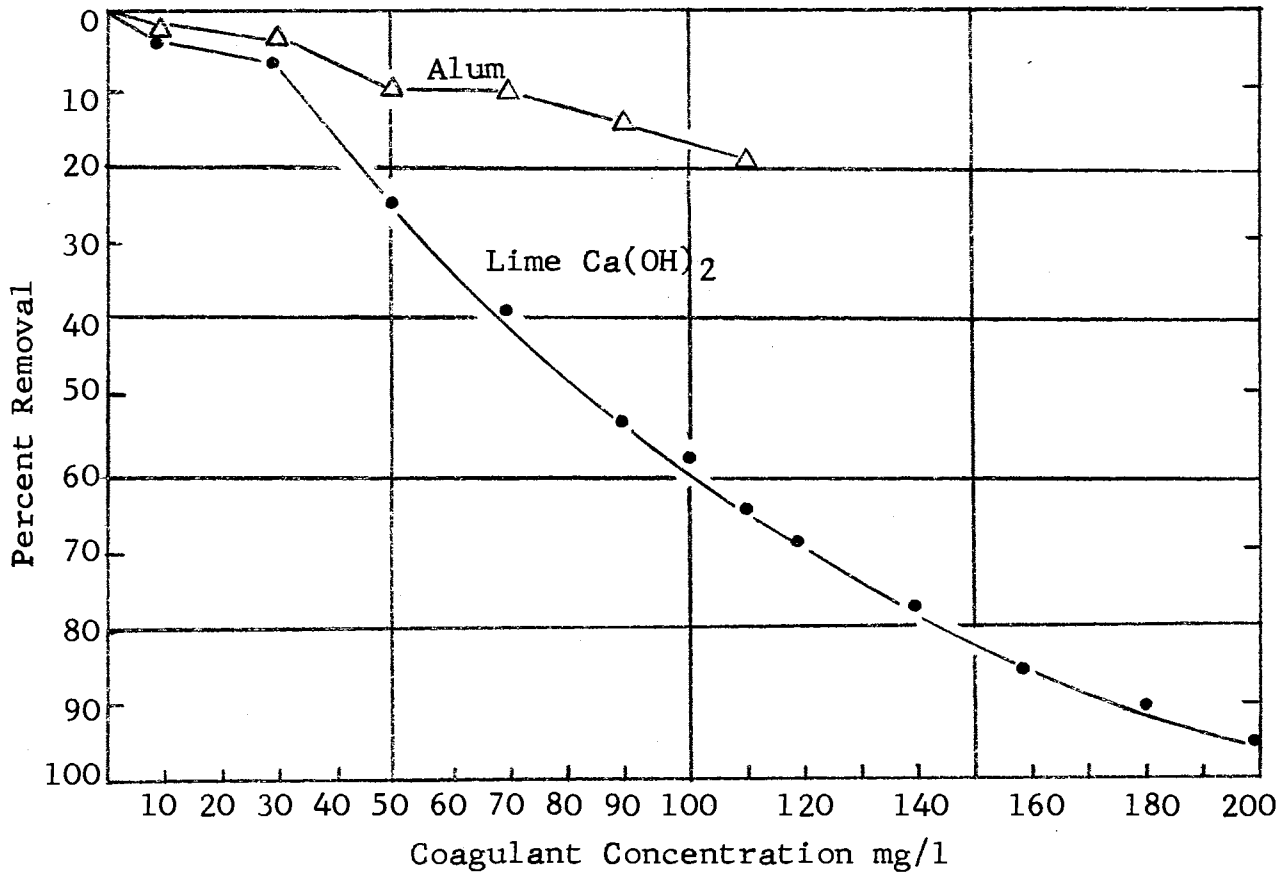

Figure 7. Phosphorous Removal By Chemical Precipitation.

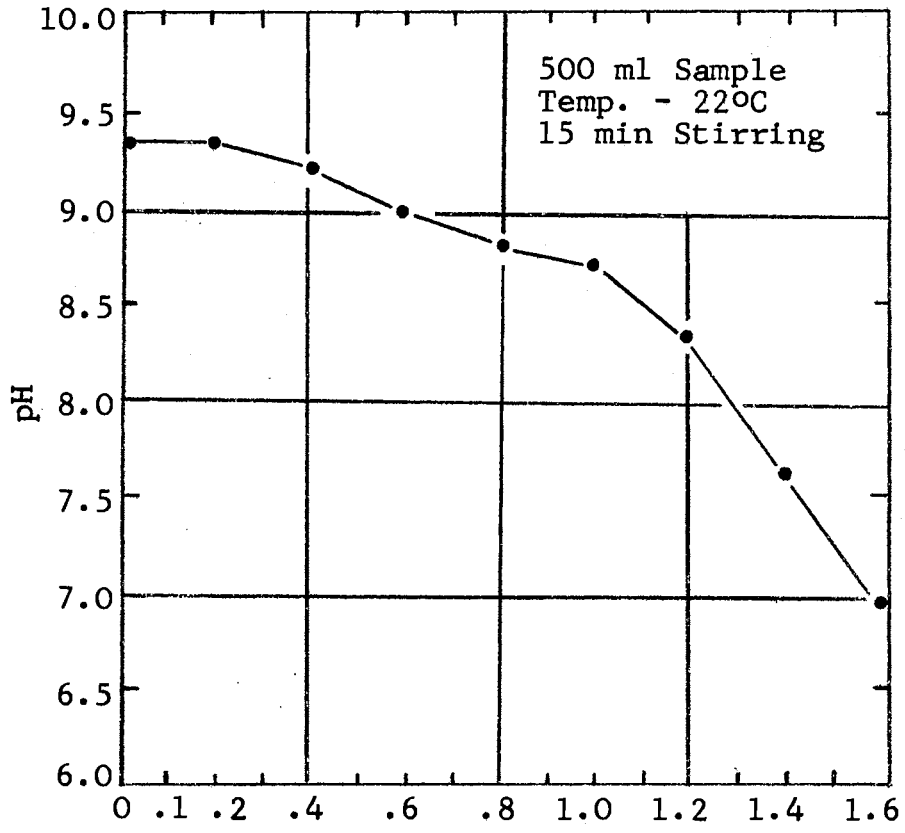

Figure 8. Titration Curve For Product Water.

final clarifier as a chemical precipitation basin also. Lime was pumped into the center well at a concentration of 200 mg/l, and an Eberbach Stirrer turning at 10-15 rpm's was used to provide slight agitation to promote flocculation. Solids were recycled in the same manner and the clarified effluent went to the storage reservoir, as usual, to be chlorinated. However, from the reservoir the water went directly to the multi-media filter. As stated, for cycle five it was decided not to use the activated carbon and see what effect this would have.

F. Experimental and Analytical Procedure

For Phase One of the study the water was sampled and analyzed at the influent to the reactor, referred to as influent (I); at the clarified effluent point, referred to as effluent (E); and after filtration, referred to as the product (P). For Phase Two, sampling and analysis were conducted at the influent; the effluent; after filtration, this time referred to as after filtration (AF); after carbon adsorption (AC); and after ion exchange, referred to as the product.

The analyses described in the following paragraphs were conducted at each of the sampling points for both phases. The standard COD and dilute COD were determined according to Standard Methods (47). The dilute COD was run in order to more accurately determine the COD for values below 50 mg/l. In addition both a total COD and a filtrate COD were run on the effluent using a 0.45 micron Millipore

membrane filter to obtain the filtrate sample. This permitted determination of both the soluble COD remaining in the clarified effluent and the COD remaining that was due to the biological solids in the clarified effluent.

Total dissolved solids were determined according to Standard Methods (47). The evaporative temperature used was 103-105°C and 100 ml samples were used.

Chlorides were determined according to the Argentometric Method outlined in Standard Methods (47).

Total phosphorous was determined according to the procedure given on pages 223-232 in the manual "FWPCA Methods for Chemical Analyses of Water and Wastes" (48).

Nitrate-nitrogen was determined for cycle one of Phase One by the Brucine Method outlined in Standard Methods (47). Since there was no nitrate added in the wastewater and none was detected in the effluent or the product, it was decided not to analyze for nitrate during the remainder of the study.

Ammonia-nitrogen was determined by a method developed by Niss and described by Ecker and Lockhart (49). Two reagents were employed. Reagent A contained: 4.7 grams sodium citrate, 1.7 grams citric acid, 9.6 grams phenol and distilled water to 480 ml. Reagent B contained: 6.0 grams boric acid H_3BO_3, 8.0 grams sodium hydroxide, 30.0 ml of commercial Chlorox bleach, and distilled water to 200 ml. Cell-free samples were diluted if needed to give between 2 and 20 mg/l of NH_3-N. To 1.0 ml samples were added 5.0 ml

of reagent A and 2.0 ml of reagent B. The samples were mixed, heated in a boiling water bath for 5 minutes, and cooled repidly in ice water. The optical density for the sample was then determined at a wavelength of 615 milli-microns against a distilled water-reagent blank using a Bausch and Lomb 120 Spectrophotometer. The optical density readings were compared to a standard curve with known concentrations of NH_3-N.

The pH was taken at each sampling point using a Beckman pH meter.

Suspended solids concentration was determined for the clarified effluent and the Mixed Liquor Suspended Solids (MLSS) in the reactor using 0.45 micron Millipore membrane filters and applying the technique outlined in Standard Methods (47). Suspended solids were determined at other points when it was considered necessary.

The transmittance was determined for all sampling points except the influent using a Bausch and Lomb 120 Spectrophotometer set at a wavelength of 450 milli-microns. The transmittance was compared against a distilled water blank.

CHAPTER IV

RESULTS

A. Phase One

Results of Phase One of the study are shown in Figures 9 through 14. The results are plotted for each cycle and lines were drawn between the values plotted to indicate any trends or expected range of values.

Figure 9 shows the COD concentration at the sampling points for each cycle. Also plotted is the effluent suspended solids concentration to show its effect on the total effluent COD as compared to the filtrate effluent COD. From Figure 9 it is seen that there may have been a slight build up of COD in the product water, but this can not be definitely concluded. Average removal obtained from influent to clarified effluent was 90 percent and from influent to product water was 96 percent. Soluble effluent COD comprised roughly 50 percent of the total effluent COD. It is interesting to see that chemical precipitation and filtration was effective in removing not only a substantial portion of the total effluent COD, but also in removing roughly 15 mg/l of soluble effluent COD. However, there remains 15-30 mg/l of COD in the product water that must be removed by other processes.

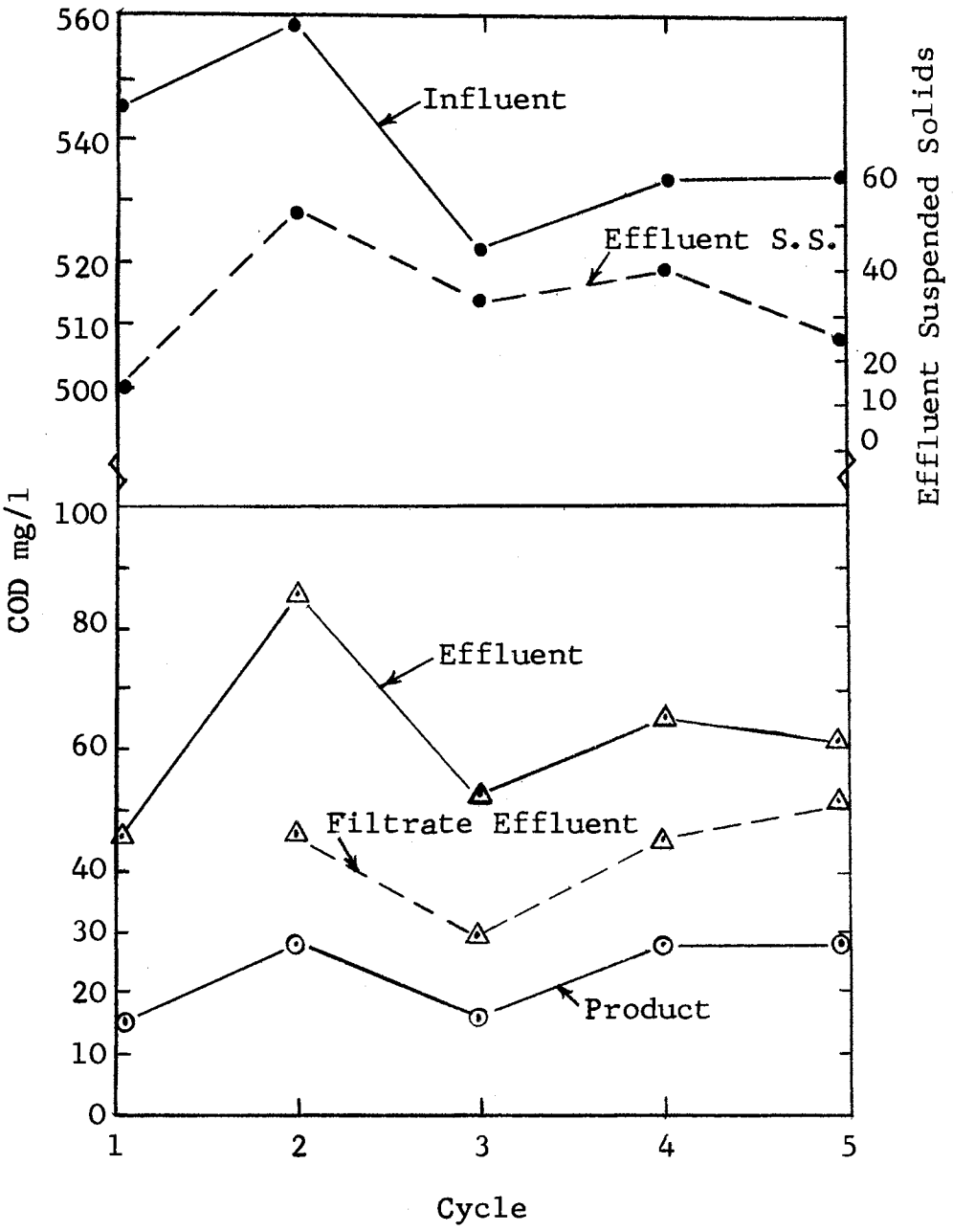

Figure 9. Chemical Oxygen Demand and Effluent Suspended Solids Phase One.

The TDS of the synthetic wastewater, i.e., the influent, was initially 1350 mg/l as shown in Figure 10. The influent to cycle five had a TDS concentration of 2500 mg/l. This gave an average incremental increase of 290 mg/l per cycle. The product water for cycle one had a TDS concentration of 645 mg/l which is above USPHS Drinking Water Standards of 500 mg/l TDS. By cycle four the TDS concentration of the product had increased to 1570 mg/l. For cycle five it is seen that both the effluent and the product showed a decrease in TDS from that of cycle four. Since the decrease in TDS showed up in the effluent it is apparent that there was an excessive uptake of TDS in the reactor. The other analyses also reveal this excessive uptake for cycle five.

The chloride concentration, as shown in Figure 11 for the influent, effluent, and product, increased in a near linear fashion for the first 4 cycles. For cycle five the incremental increase in the influent was slightly less than for the other cycles, but both the effluent and the product showed an unexpected decrease.

Figure 12 shows how the total phosphorous concentration varied for each cycle. As previously stated, the lime concentration added for chemical precipitation was based on at least 90 percent removal of the phosphorous remaining in the effluent. However, the percent removal steadily decreased from 98 percent for cycle one to 78 percent for cycle four. This steady decrease in removal capability can

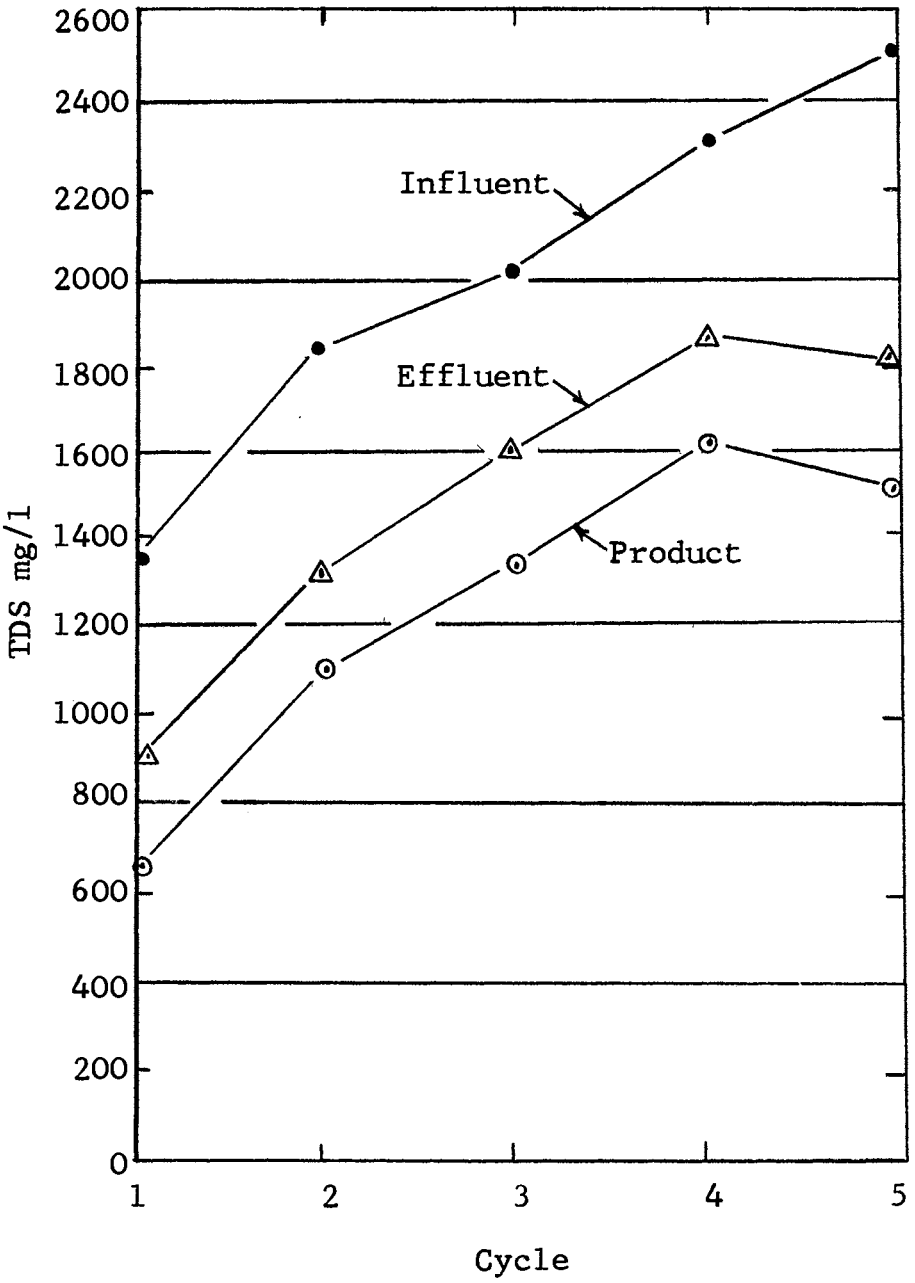

Figure 10. Total Dissolved Solids Phase One.

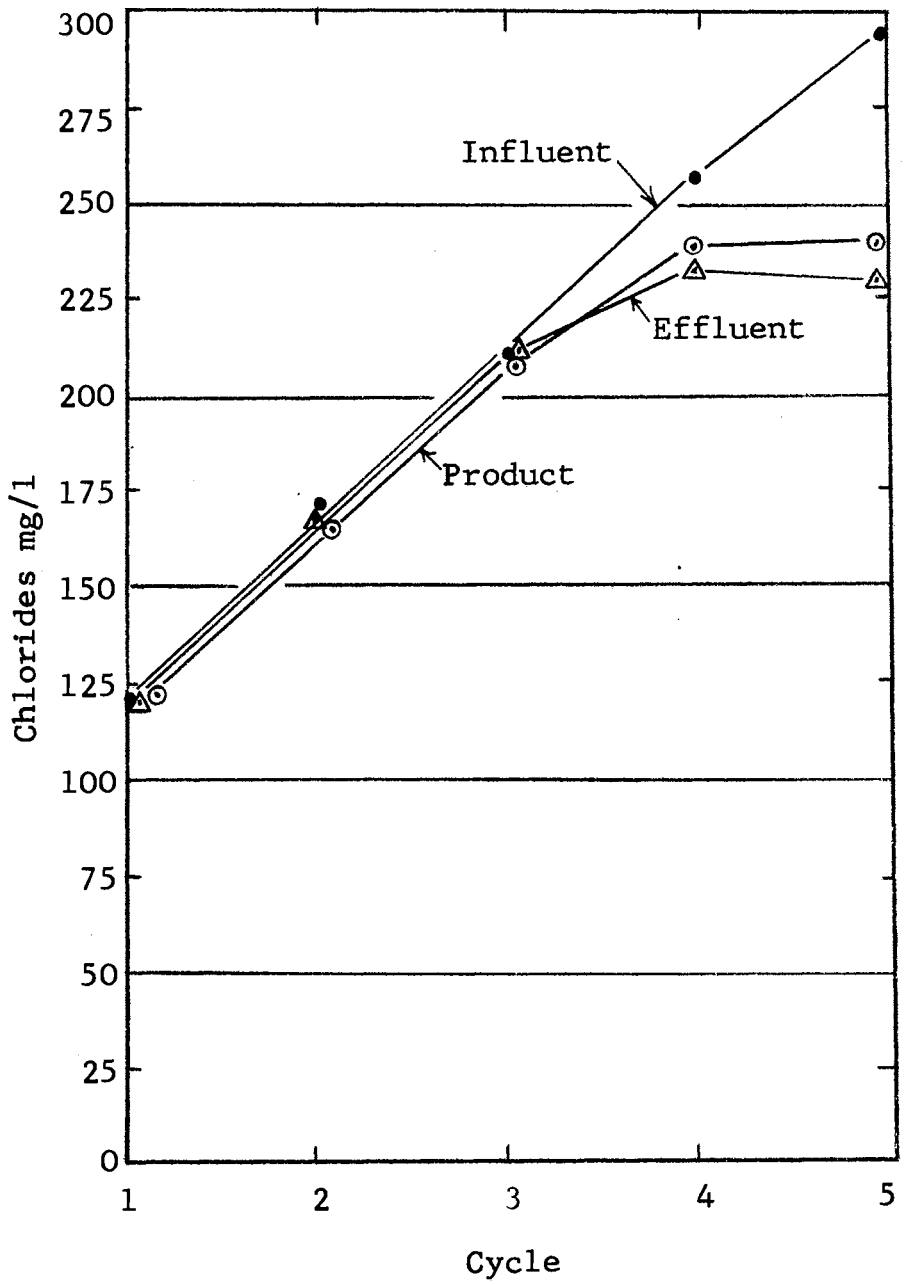

Figure 11. Chlorides Phase One.

not be explained by a decrease in the pH obtained, which as explained before is the primary parameter affecting phosphorus removal, because the pH did not steadily decrease. The product water pH for cycle one was 9.5, for cycle two 8.6, for cycle three 9.0, and for cycle four 8.9. Uptake of phosphorous in the reactor varied from a low of 4 mg/l or 4 percent to a high of 22 mg/l or 21 percent. This uptake, or more precisely difference in uptake, seemed independent of the MLSS concentration in the reactor, since the highest uptake occurred at the lowest MLSS concentration.

The ammonia-nitrogen concentration for each cycle is shown in Figure 13. Like other compounds analyzed for, there is a general build up through direct reuse of the water. Again the excessive uptake was noted for cycle five.

Turbidity and transmittance are shown in Figure 14. The turbidity as expressed in Jackson Turbidity Units (JTU) was related to the transmittance by a table obtained from the Stillwater, Oklahoma water treatment plant. This table is based on experimantal correlation between JTU and transmittance determined at a wavelength of 450 milli-microns using 1 inch tubes. It is not known for certain that the correlation obtained, from which the table was made, would be roughly the same as that obtained if the wastewater in this study was used, but for presentation purposes both the transmittance and the JTU are shown. Transmittance was determined after chemical precipitation also, but is not

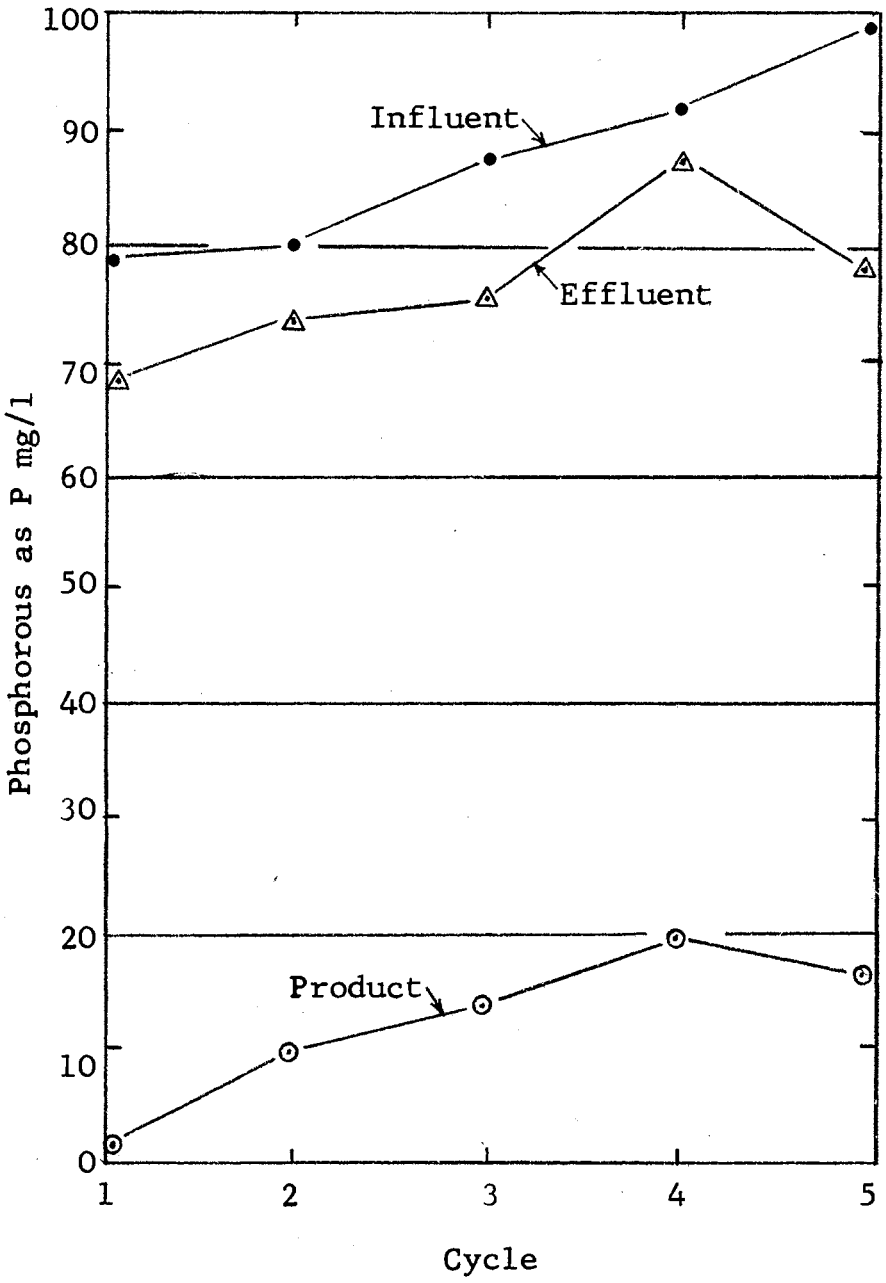

Figure 12. Total Phosphorous Phase One.

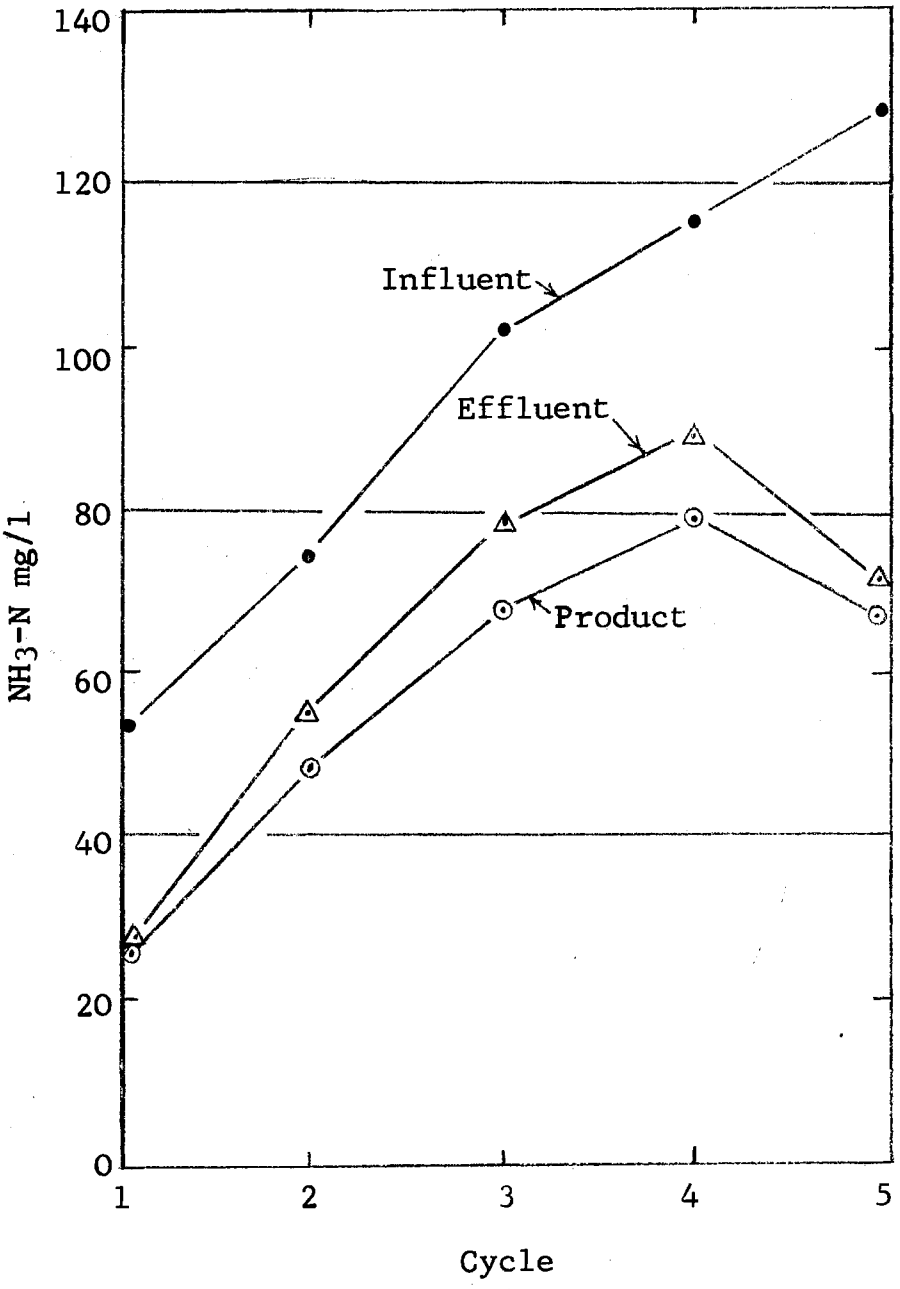

Figure 13. Ammonia-Nitrogen Phase One.

shown in Figure 14 because it was considered to be too dependent on how carefully the water was decanted from the precipitation beakers. In general the transmittance after precipitation was 4-6 percent less than that of the product water indicating that the multi-media filter was removing turbidity causing material.

During cycle three there was a marked change in predominance of the microorganisms within the reactor, and this effect showed up in the effluent and the product water in the form of a yellow green color. This yellow green color remained through the next two cycles. The visual difference between the product water and distilled water was much more than the transmittance tended to indicate. Suspended solids in the product water were not detectable by the membrane filter technique, so it was considered that the color was a soluble product from the microorganisms in the reactor. With the exception of cycle three there was a good correlation between the effluent suspended solids shown in Figure 9 and the transmittance obtained for the effluent.

Other information determined but not plotted is that of the pH and the MLSS concentration in the reactor. The pH of the effluent varied between 6.5 and 7.0. For the product it varied from 8.6 to 9.5. As stated earlier the pH was neutralized to 7.6 before reuse. It was desired to maintain the MLSS concentration in the reactor between 2000 and 2500 mg/l. In practice it varied from roughly 1600 to

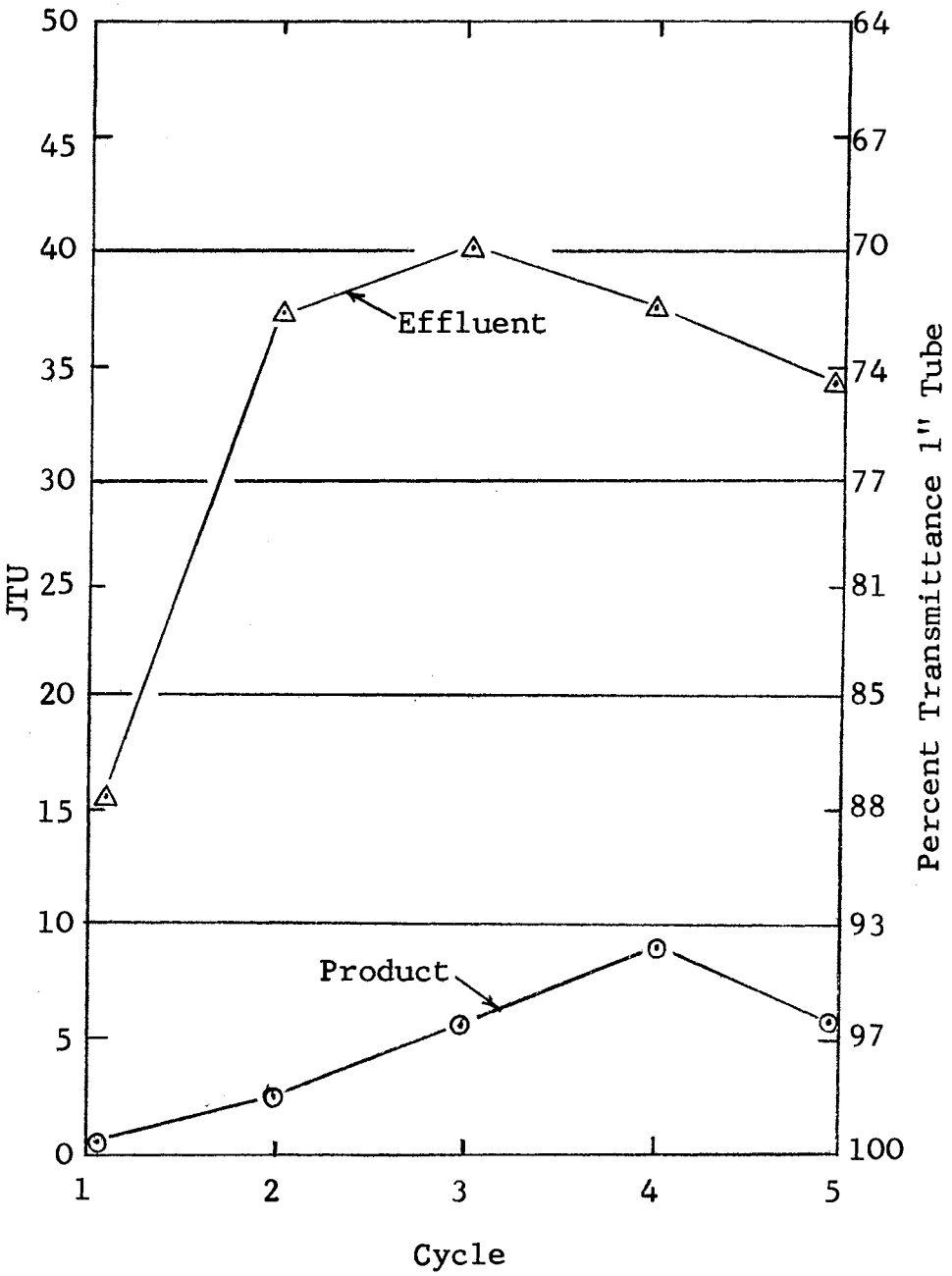

Figure 14. Turbidity and Transmittance Phase One.

2350 mg/l for Phase One.

B. Phase Two

Results of the analyses for Phase Two are presented in Figures 15 through 20. Like those presented in Phase One, the purpose of these figures are to present a visual representation of any build up trends and to define the range of expected values one might encounter through continuous reuse of the water.

Figure 15 presents the data obtained for COD during Phase Two. Also presented is the effluent suspended solids concentration. Before attempting to describe the results it should be noted that during the shut down time between cycle two and three the main air line failed, cutting the air supply, and was not detected for a period of hours. The length of time that the reactor was without air was sufficient to cause a marked change in predominance of the microorganisms within the reactor. This led to non-flocculent conditions as evidenced by 410 mg/l of suspended solids in the clarified effluent, a pH of 4.5 in the effluent, and foaming in the reactor. Cycle three was continued, but after seeing the results it was decided to take corrective measures. The reactor was emptied, reseeded, and batch fed for a period of approximately 18 days until flocculent conditions were again obtained.

Cycle four was then started. The water from cycle three was stored in closed reservoirs for the 18 day period without any noticable stagnation. It should be mentioned

again that cycle four was conducted without using the jar test apparatus for separate chemical precipitation, but rather it was attempted to incorporate the final clarifier as the chemical precipitation basin. Also, cycle five was conducted without using activated carbon adsorption.

The total effluent COD was rather high for all but the first cycle. This is primarily due to the high effluent suspended solids concentration. Separate chemical precipitation, i.e., cycles one, two, three, and five, was able to decrease the COD to a more respectable concentration as seen by the after filtration values. This was accomplished by removing virtually all the suspended solids and a portion of the soluble effluent COD. The activated carbon was able to decrease the residual COD to 1.1 mg/l for cycle one, but could not achieve such good removal for the next two cycles. However, the residual COD after cycle three was still below 10 mg/l. In making calculations assuming an adsorptive capacity of 0.5 pounds COD per pound of carbon, the activated carbon should not have approached exhaustion. It should be remembered that in terms of contact time the column was definitely underdesigned.

For cycle four, utilizing the final clarifier as the chemical precipitation basin also, did not work in terms of removing the effluent suspended solids. Filtration was not successful in removing the suspended solids completely although it did decrease the concentration from 150 to

50 mg/l. The 50 mg/l of suspended solids after filtration explains why the after filtration COD is higher than the soluble effluent COD. The suspended solids concentration was 32 mg/l after carbon adsorption which is why the after carbon COD is so high. Suspended solids carried through the ion exchanger, but by this time the concentration was less than 20 mg/l.

It is interesting to see that for cycle five without activated carbon there is a substantial decrease in COD of 20 mg/l between the after filtration value and that of the product water. There were no detectable suspended solids after filtration or in the product water. This indicates that some of the residual COD is ionized and can be removed by ion exchange.

TDS concentration variation is shown in Figure 16. As seen there was no apparent build up of TDS at any of the sampling points. This was due primarily to the ion exchange unit which was capable of reducing the TDS to less than 25 mg/l. The reason that cycle one values are highest for I, E, AF, and AC is because the wastewater was made using tap water which had 250-350 mg/l of TDS, whereas the wastewater for cycles two-five was made with the reused demineralized water.

Figure 17 is a plot of the chloride concentration for each cycle. As seen it is remarkedly different than the chloride concentration plot of Figure 11 for Phase One. There is definitely no build up of chlorides for Phase Two

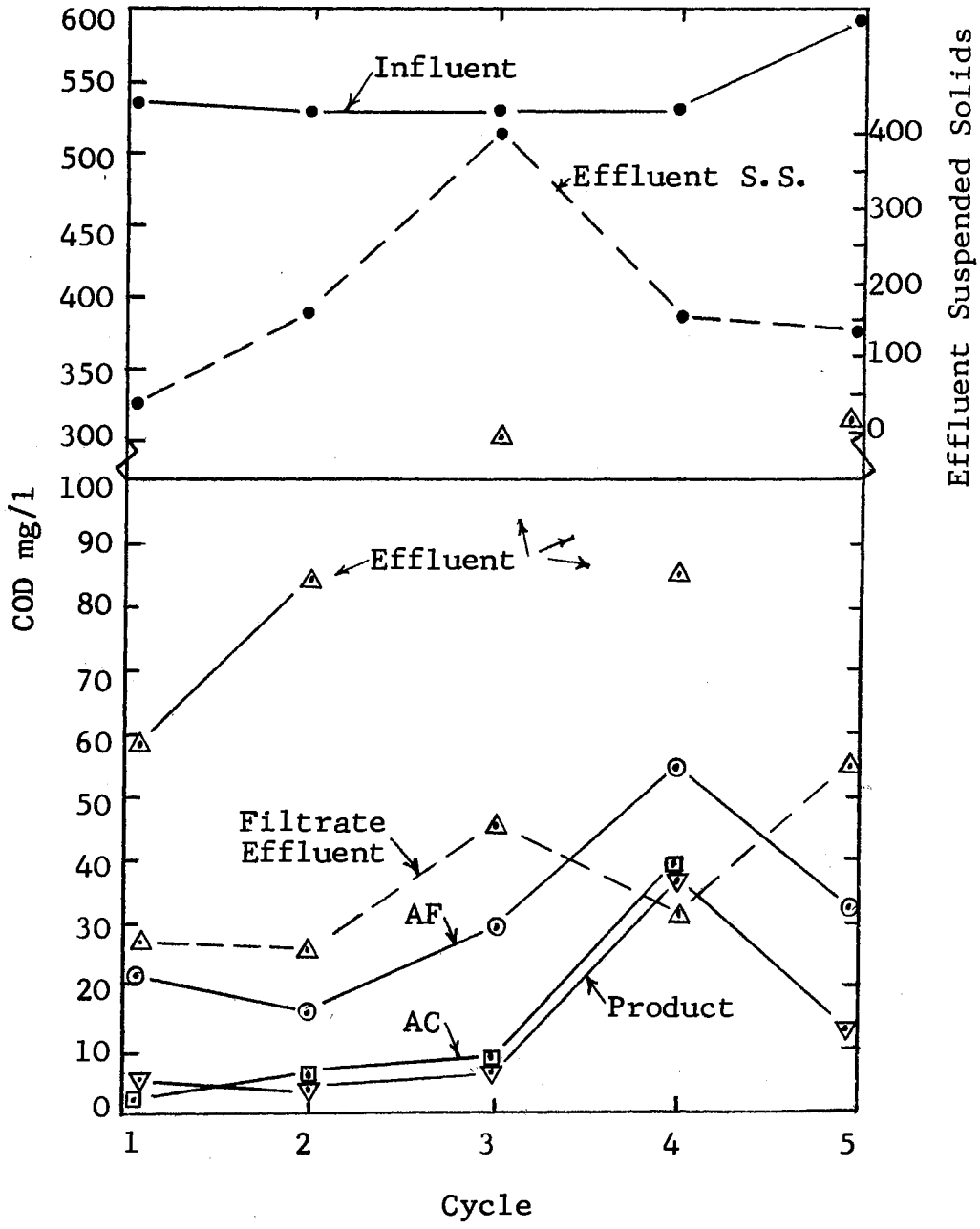

Figure 15. Chemical Oxygen Demand and Effluent Suspended Solids Phase Two.

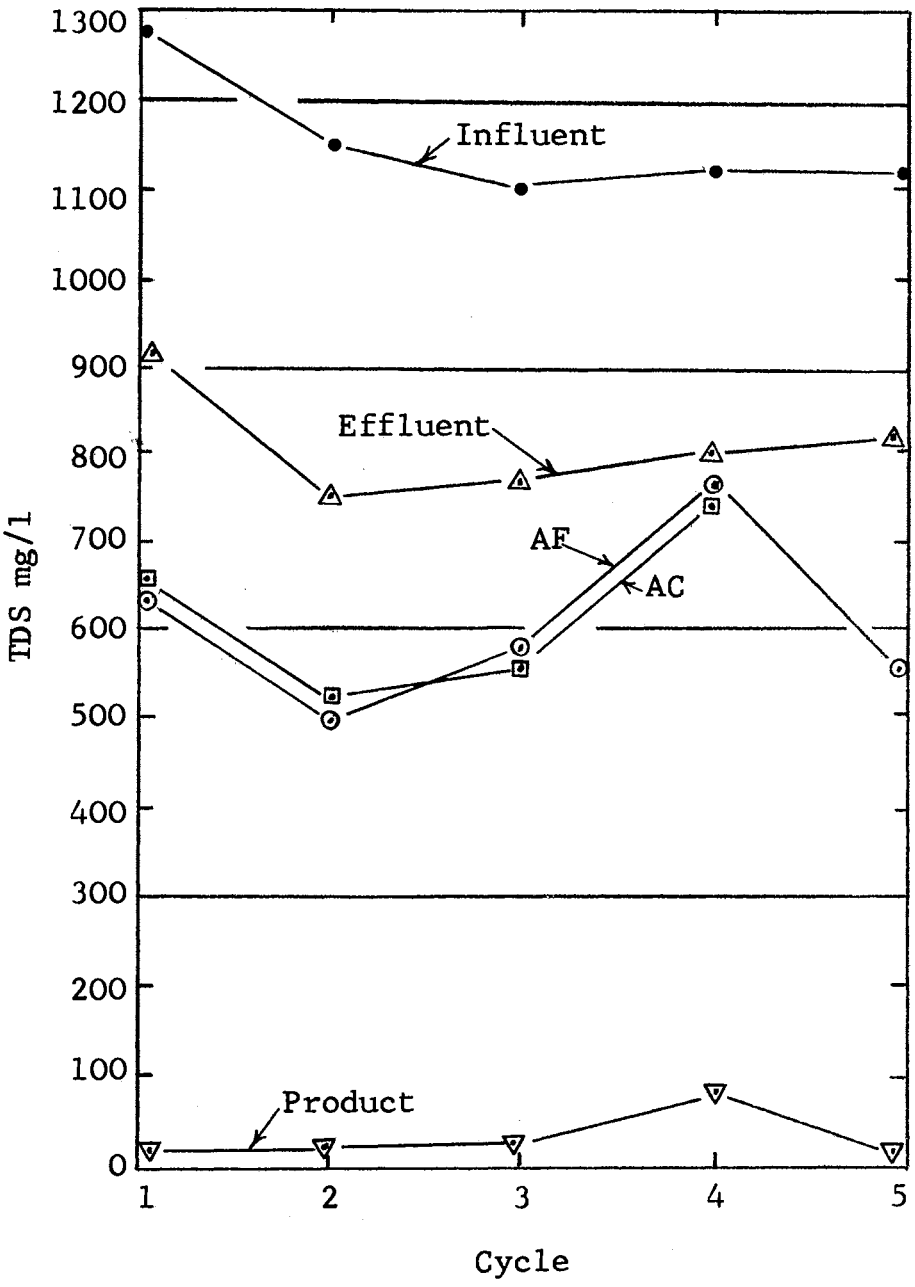

Figure 16. Total Dissolved Solids Phase Two.

due to the ion exchange unit which removed essentially all of the chlorides. As was explained earlier, cycle one values are higher due to using tap water to make up the wastewater initially. The chloride concentration of the tap water was in the range of 50-70 mg/l. It is interesting to note that for the method used to analyze for chlorides, the milliliters of titrant required were less for the product water than for the distilled water blank. This possibly indicates that the ion exchange unit was more efficient in removing chlorides than was distillation.

Total phosphorous concentration is shown in Figure 18. There appears to be a build up in the influent, but it is considered that this is not a build up, but rather is due to variations in the chemical (feed) addition since the product water was virtually free of phosphorous. The effluent for cycles three and five had a higher phosphorous concentration than the influent. This is entirely possible as microorganisms in an activated sludge unit have been known to undergo periods of excessive phosphorous uptake and periods of phosphorous release. It is noted that the removal efficiency of chemical precipitation decreased for cycles two and three in comparison to cycle one. In contrast to the explanation offered in Phase One, it is considered that this time the decreased efficiency could have been caused by lower obtainable pH values. For cycle one the pH after precipitation was 9.4, for cycle two it was only 8.8, and for cycle three only 8.3.

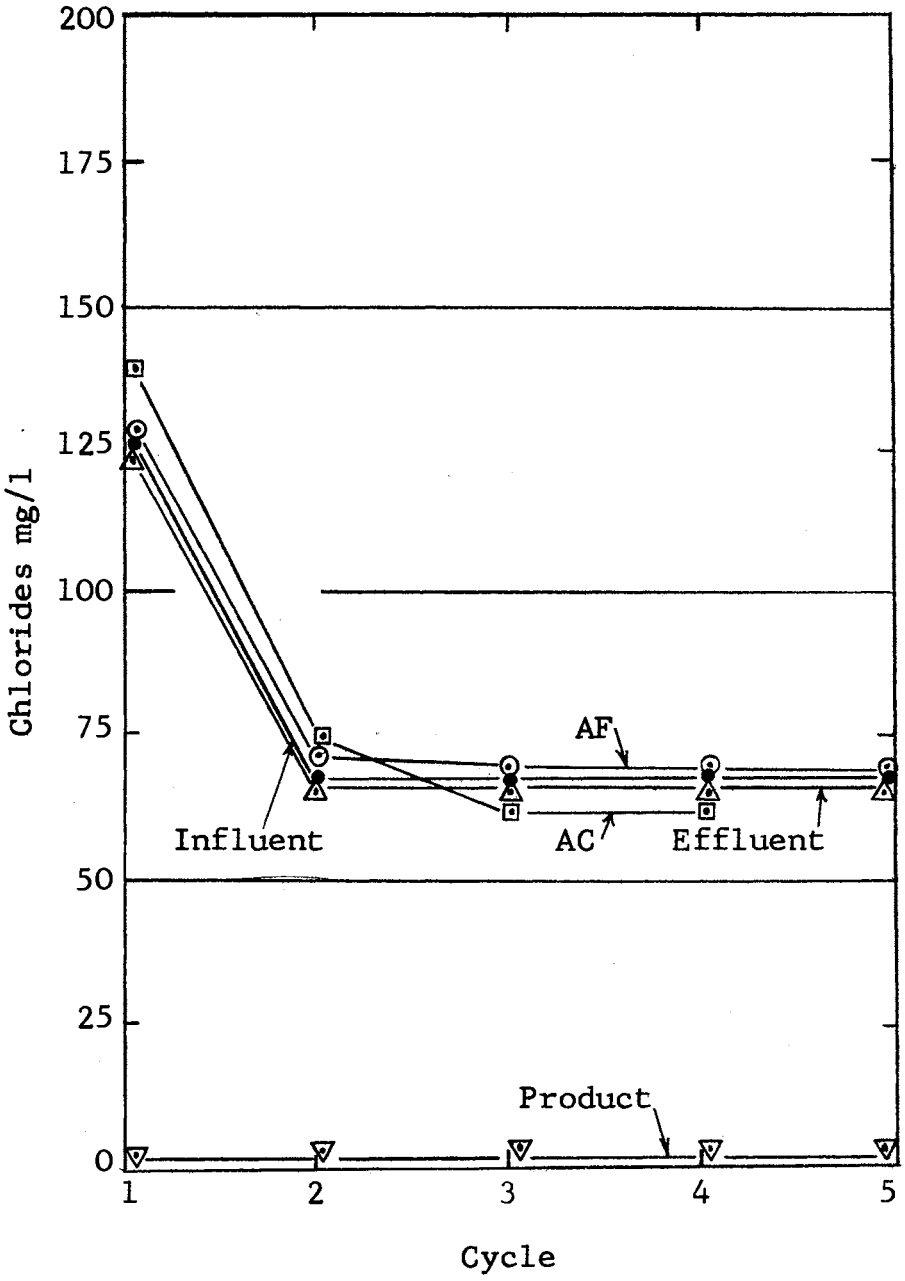

Figure 17. Chlorides Phase Two.

Considering cycle four it is seen that the modification to the treatment process was ineffective in precipitating phosphorous. It is beleived that this is primarily due to only being able to obtain a pH of 7.9 in the effluent.

Figure 19 expresses the ammonia-nitrogen concentration for each cycle. As expected there is fluctuation between cycles but there is no indication of a build up. The ion exchange unit is seen to be very effective in removing ammonia-nitrogen. It is considered that the after carbon adsorption value for cycle one and the after filtration value for cycle four are slightly in error since they appear to be higher than the effluent NH_3-N value. The after carbon adsorption sample for cycle four did not develop properly during the ammonia-nitrogen test and was therefore omitted.

Transmittance and turbidity are plotted in Figure 20. Except for cycle five the effluent transmittance is roughly correlated to the effluent suspended solids concentration shown in Figure 15. The high turbidity for the after filtration sample of cycle four is due to the presence of suspended solids remaining in the water. It is seen that except for cycle two the product water always yielded 99-100 percent transmittance.

Other data that was not plotted is that of the pH values and the MLSS concentration. The pH of the effluent varied from 6.7-7.2 for cycles one, two and five. For cycle three the effluent pH dropped to 4.5 due to the upset

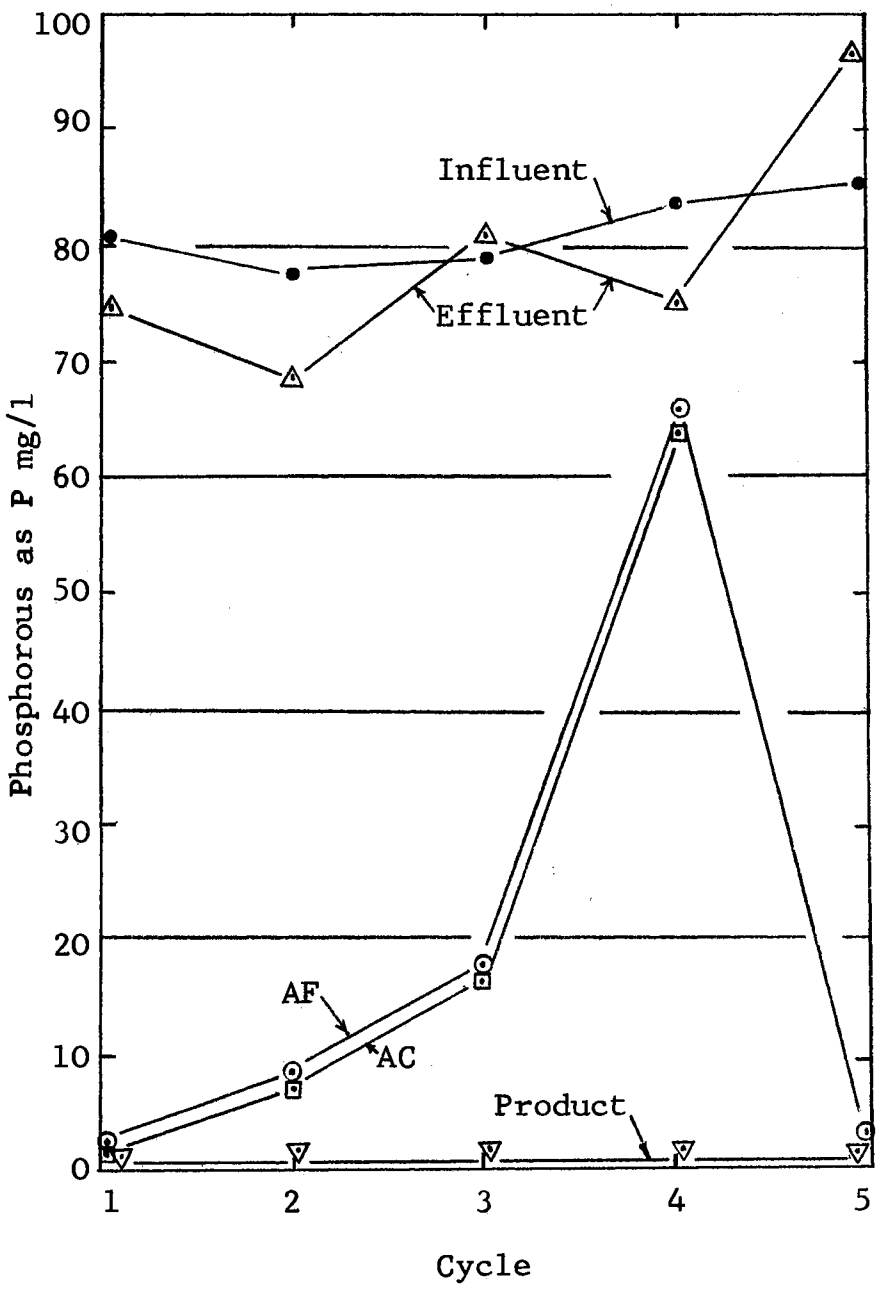

Figure 18. Total Phosphorous Phase Two.

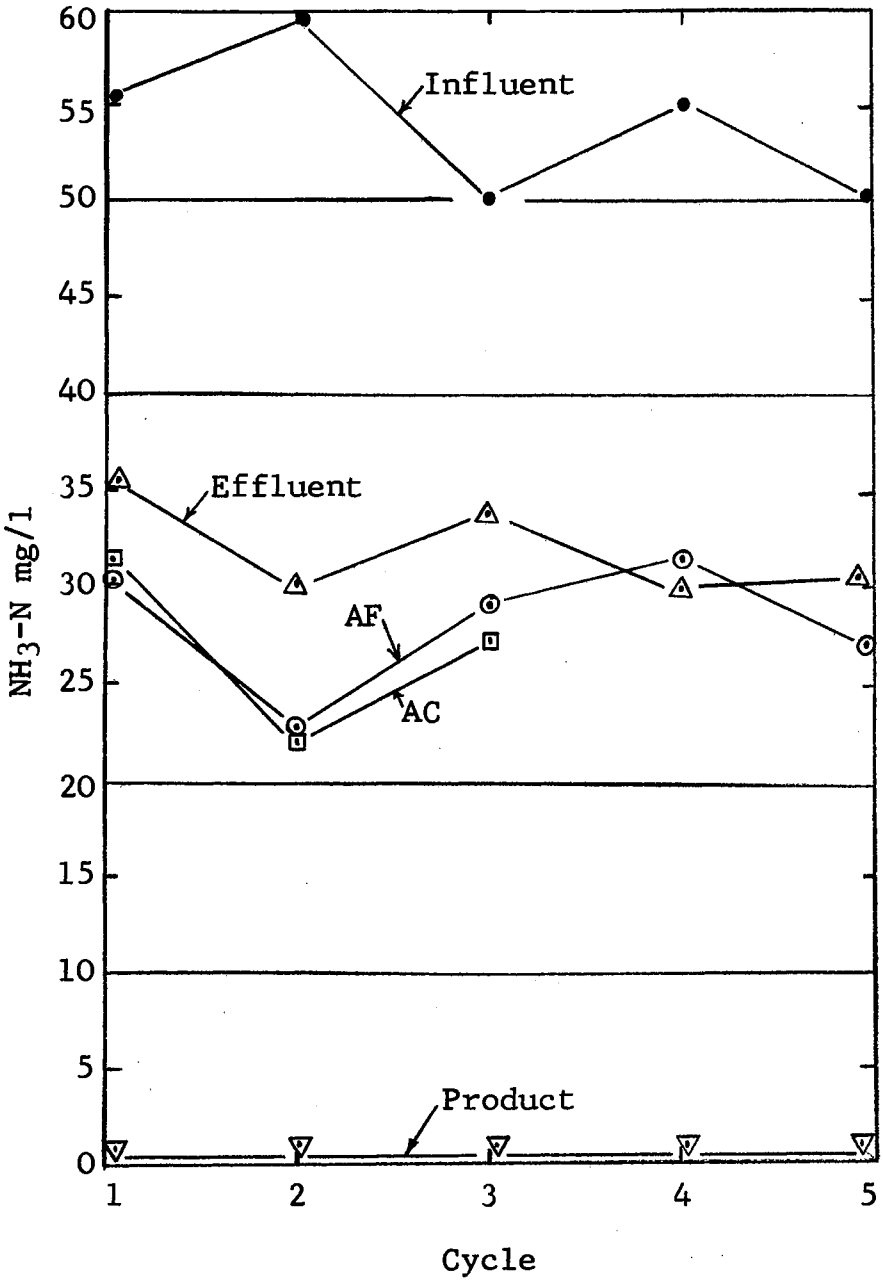

Figure 19. Ammonia-Nitrogen Phase Two.

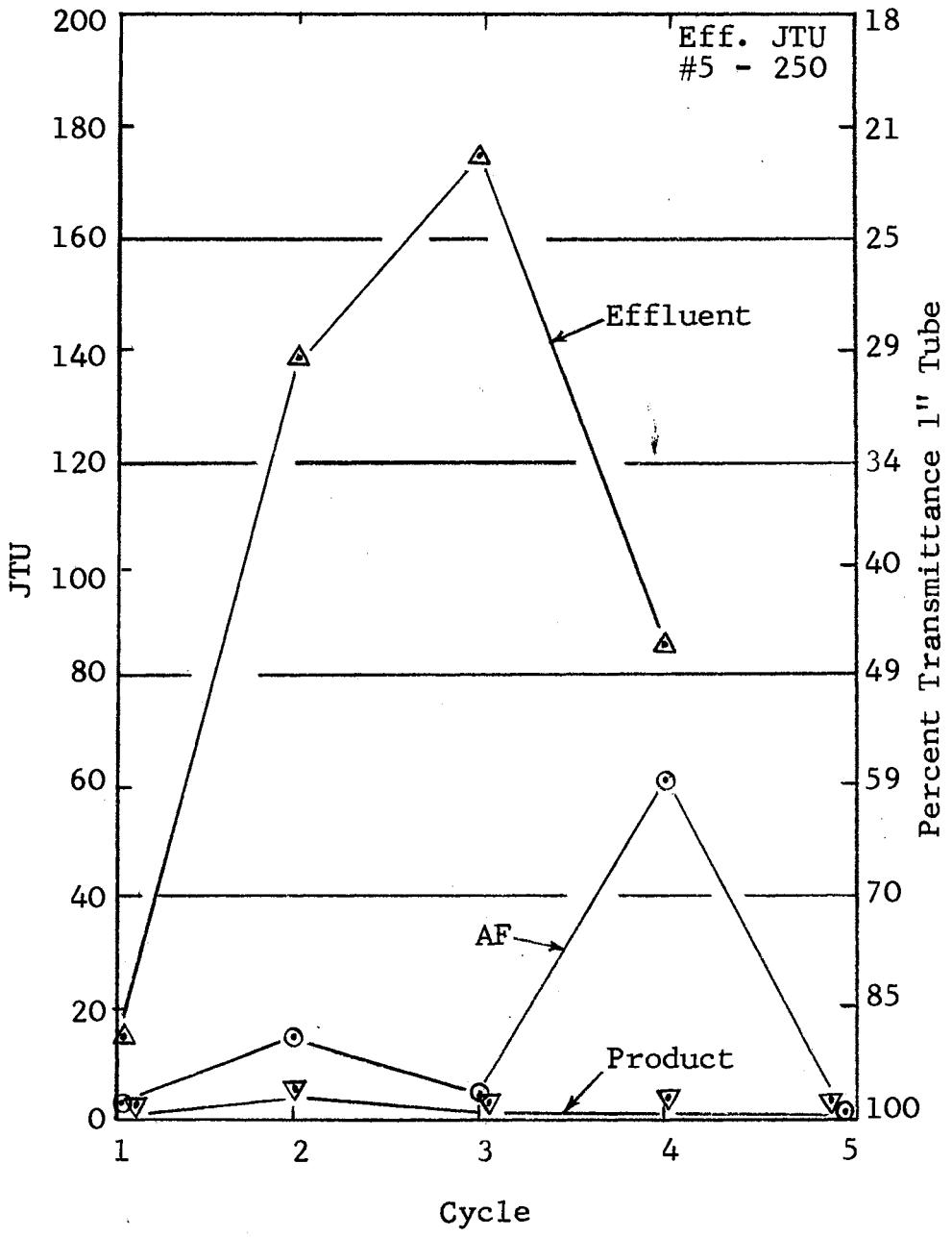

Figure 20. Turbidity and Transmittance Phase Two.

biological conditions in the reactor. For cycle four the effluent pH was 7.9 due to the treatment modification. After filtration pH values ranged from 8.3-9.4. It is interesting to note that the pH after carbon adsorption consistently decreased in comparison to the after filtration value. The pH after carbon adsorption varied from 7.2-7.4. The pH of the product water varied from 6.6-7.7 This variation is considered to have been caused by whichever exchange ion was in a higher concentration in the product water. As was stated earlier it was attempted to maintain 2000-2500 mg/l of MLSS in the reactor. However, the MLSS concentration did not get above 1450 mg/l except for cycle one where it was 2770 mg/l. The low was 1020 mg/l for cycle three.

C. Removal Capabilities of Each Unit Process

The capabilities of each unit process to remove the contaminants and characteristics analyzed for are presented in graphical form in Figures 21 through 26. Both the average of the 5 cycles for Phase Two and the optimum cycle, in terms of removal, for Phase Two are plotted. In Figure 27 the removal capability is shown in a flow diagram form. This is for the optimum cycle. It is realized that the final concentration is the most important consideration, but in discussing the figures, percent removal will be primarily utilized.

COD removal is presented in Figure 21. On the average only 70 percent of the initial COD was removed by biological

treatment which was certainly not very good treatment. Because a high percentage of the effluent COD was caused by the presence of suspended solids in the effluent it is apparent that a more positive control over the effluent suspended solids is required. The optimum cycle obtained 90 percent removal of COD which is much more respectable. Precipitation and filtration were successful in removing another 25 percent on the average and approximately another 6 percent for the optimum. Roughly 98 percent of the influent COD was removed by the time the water had gone through carbon adsorption. It must be remembered that this was obtained with an underdesigned carbon column.

TDS removal is shown in Figure 22. It is seen that biological treatment removed an average of 25 percent, precipitation and filitration another 22 percent, and ion exchange virtually all of the remaining 53 percent. It was observed that, on the average, the TDS concentration increased after carbon adsorption compared to the after filtration value. This increase was consistent and was in the range of 15 mg/l. Either this was an experimental error or possibly the activated carbon may have some ion exchange capabilities.

Chlorides as shown in Figure 23 are virtually unaffected by any treatment process except ion exchange which was very effective in removing the chloride ion. It appears that in treating the wastewater chlorides were actually concentrated.

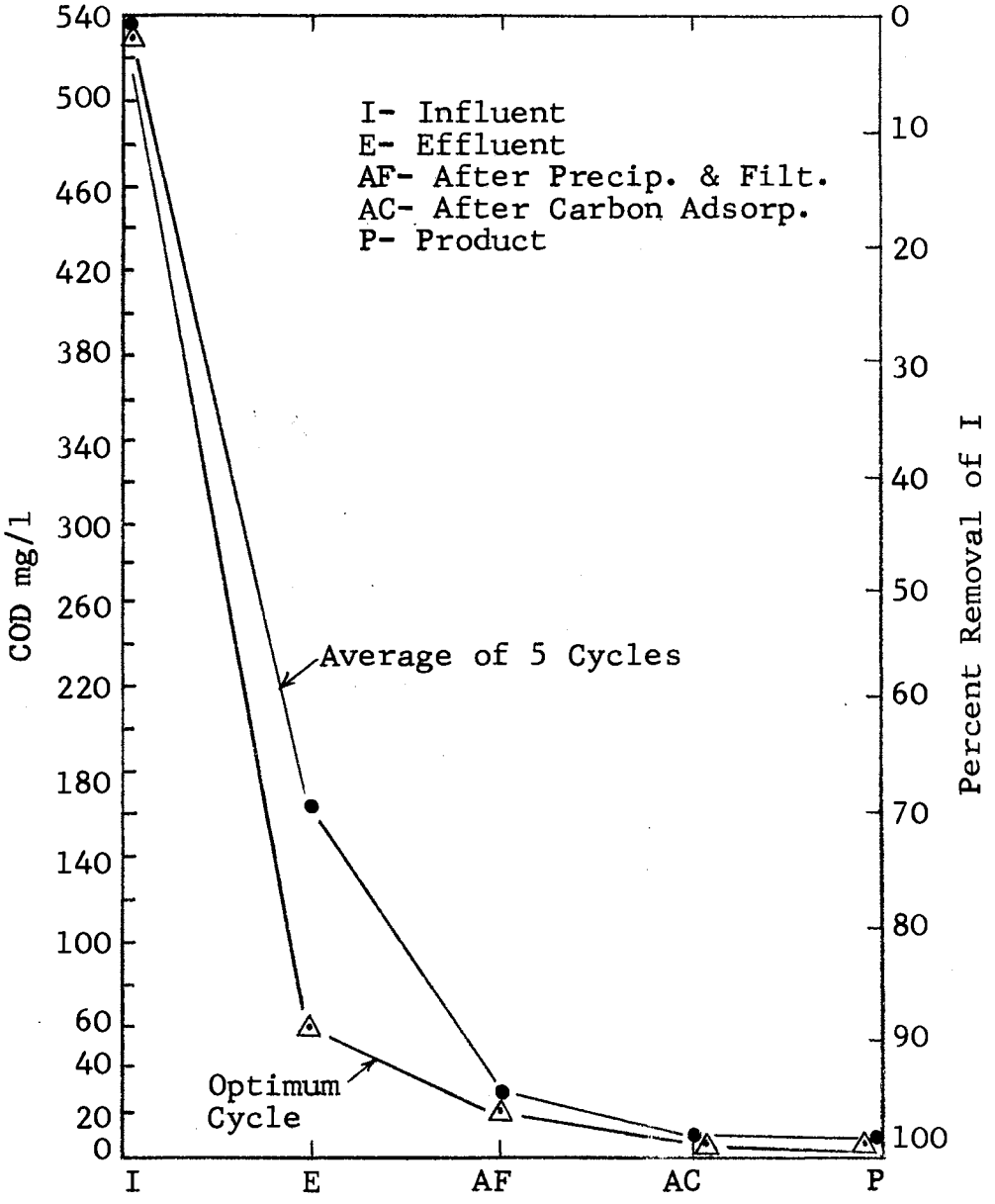

Figure 21. Chemical Oxygen Demand Removal By The Unit Processes.

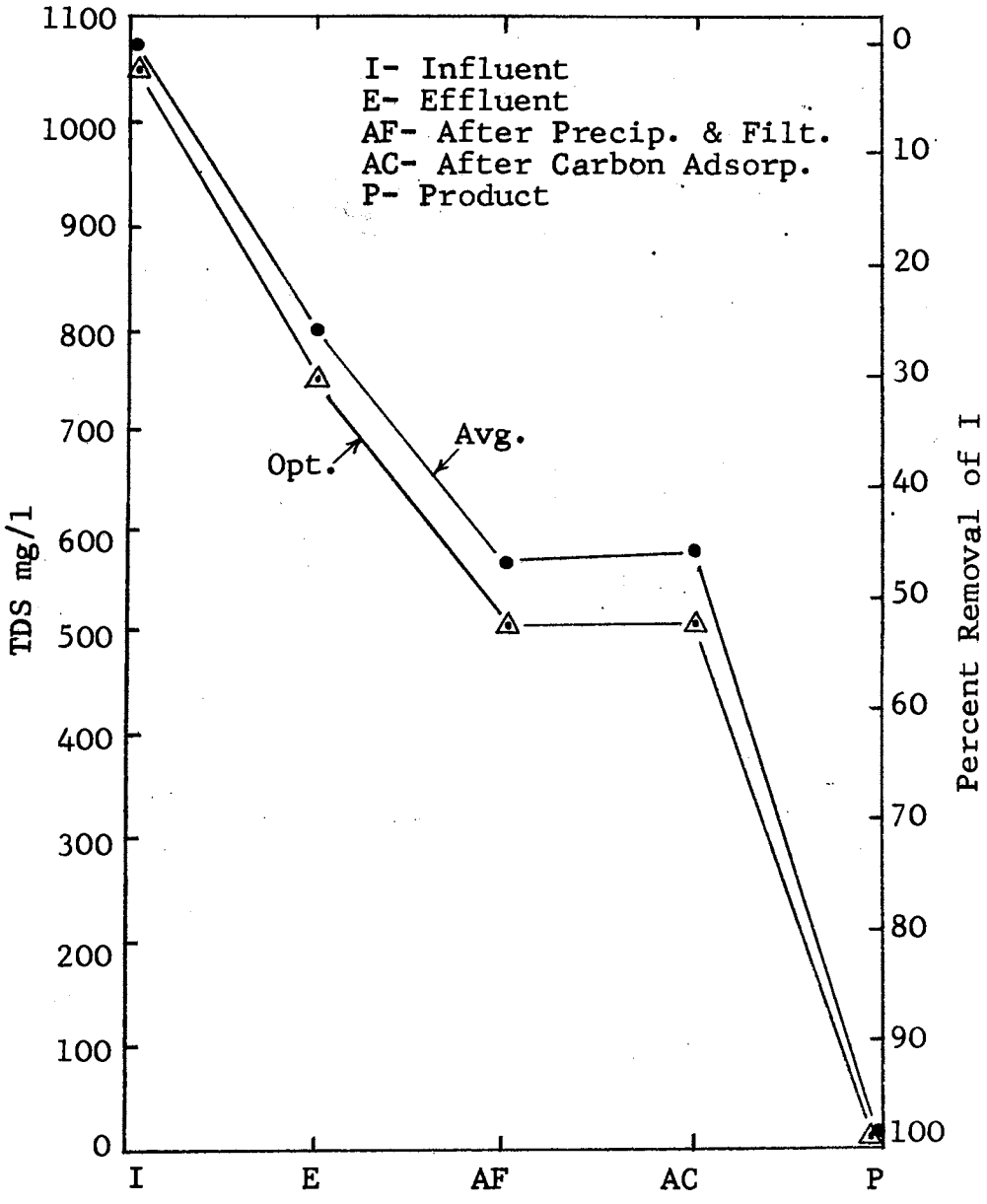

Figure 22. Total Dissolved Solids Removal By The Unit Processes.

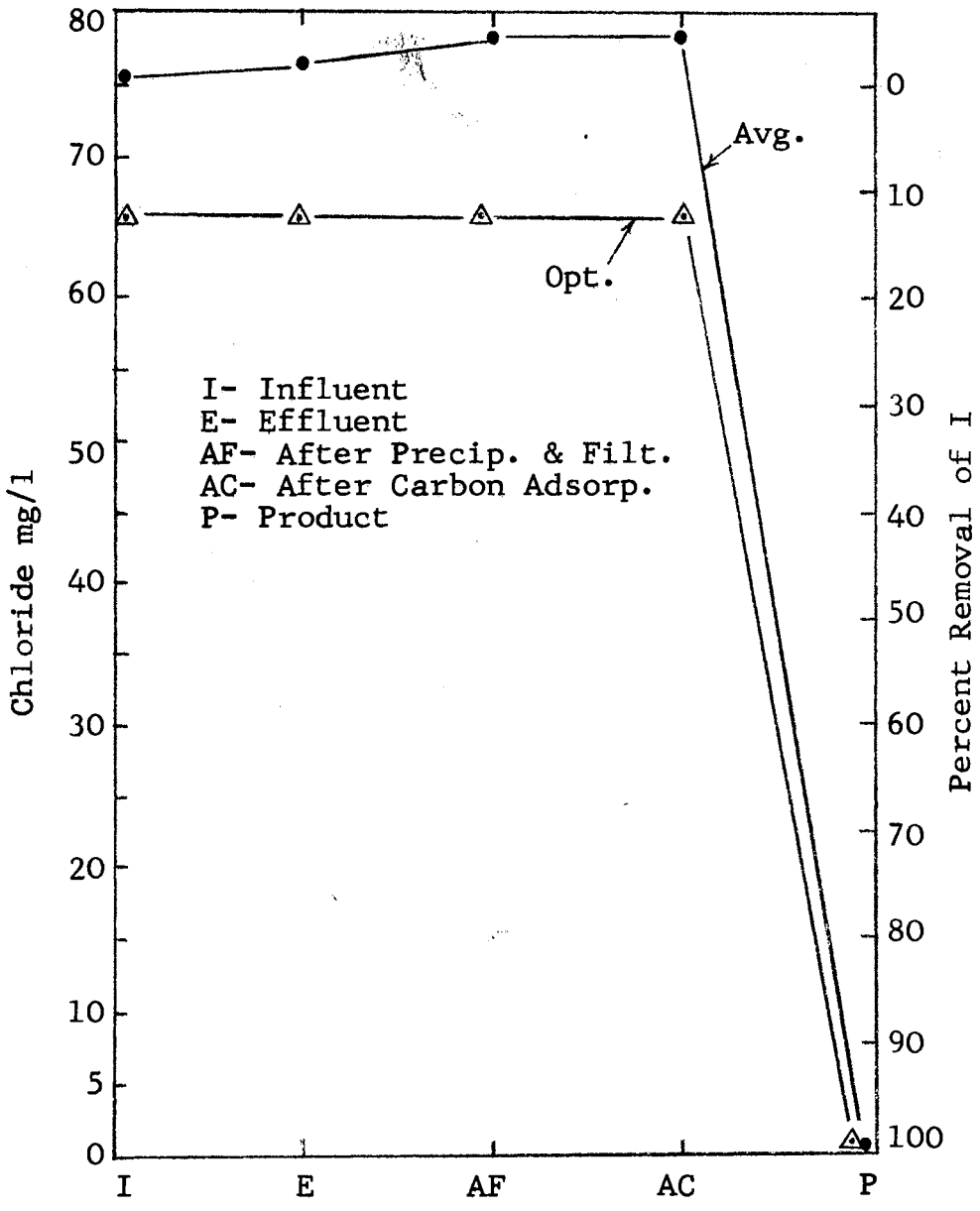

Figure 23. Chloride Removal By The Unit Processes.

Phosphorous removal is shown in Figure 24. As was stated earlier the lime dosage applied for chemical precipitation was intended to give at least 90 percent removal of the phosphorous in the effluent. As it turned out that, on the average, 91 percent of the influent phosphorous was removed by biological treatment, and precipitation and filtration (shown in Figure 24) and slightly under 90 percent of the effluent phosphorous was removed (not shown in Figure 24). This reveals that, on the average, only 1 percent of the phosphorous was removed by biological treatment. Ion exchange was effective in obtaining a phosphorous concentration of practically zero.

Figure 25 reveals how ammonia-nitrogen was removed. Roughly 40 percent was utilized in the reactor. Another 5 percent was removed by precipitation and filtration and practically all of that remaining by ion exchange. For the optimum cycle almost 20 percent was removed by precipitation and filtration.

Turbidity and transmittance are given in Figure 26. As seen there is a great difference between the average obtained and the optimum obtained especially for the effluent. However, it was still possible to obtain water with virtually the same transmittance as distilled water.

Figure 27 gives a diagramatic representation of the treatment process effectiveness. Also it shows precisely where sampling was conducted. As mentioned earlier, the values presented in this figure are those obtained for the

optimum cycle in terms of removal. The product water is virtually free of all the chemical contaminants that were analyzed for and easily meets the requirements of the USPHS Drinking Water Standards.

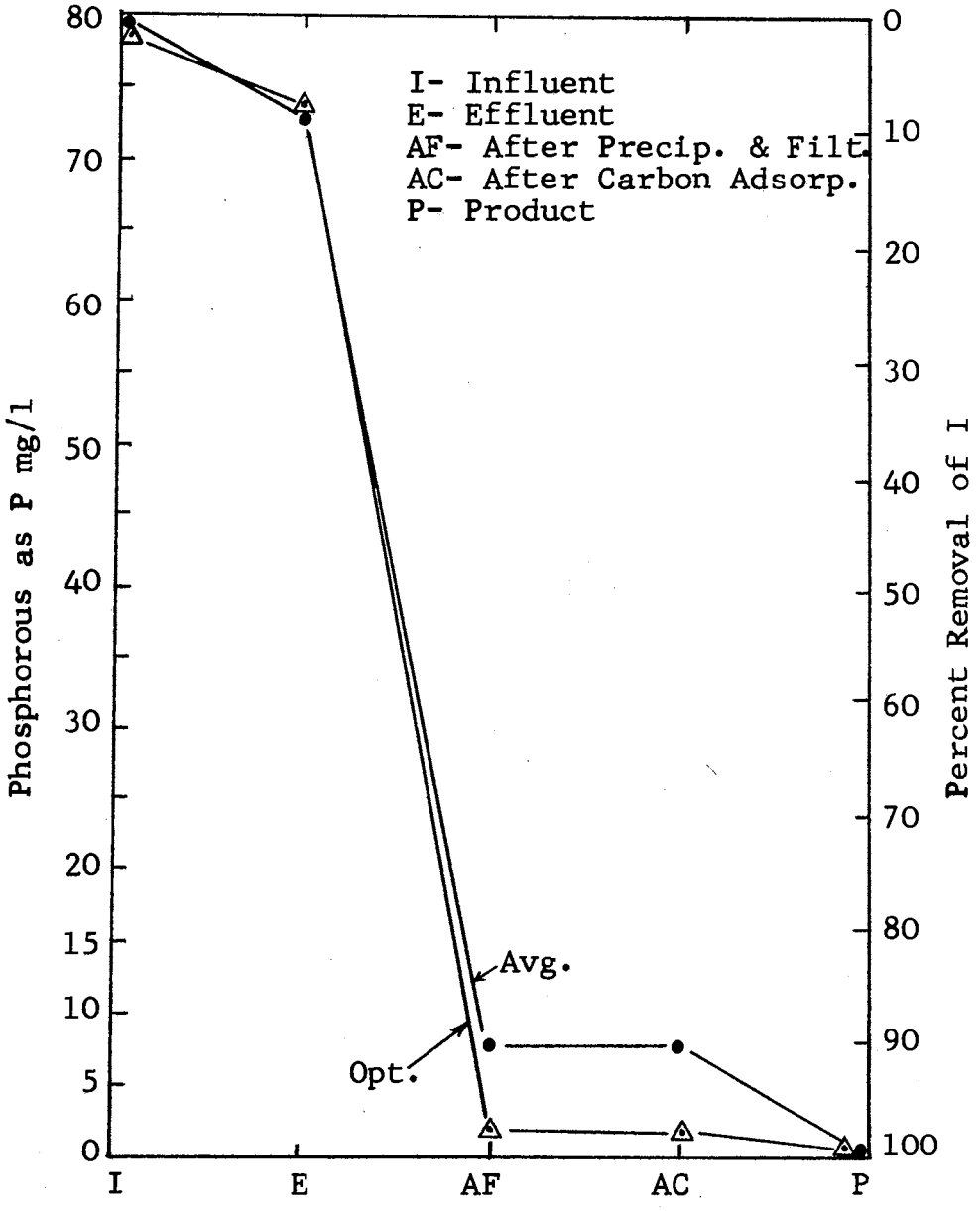

Figure 24. Total Phosphorous Removal By The Unit Processes.

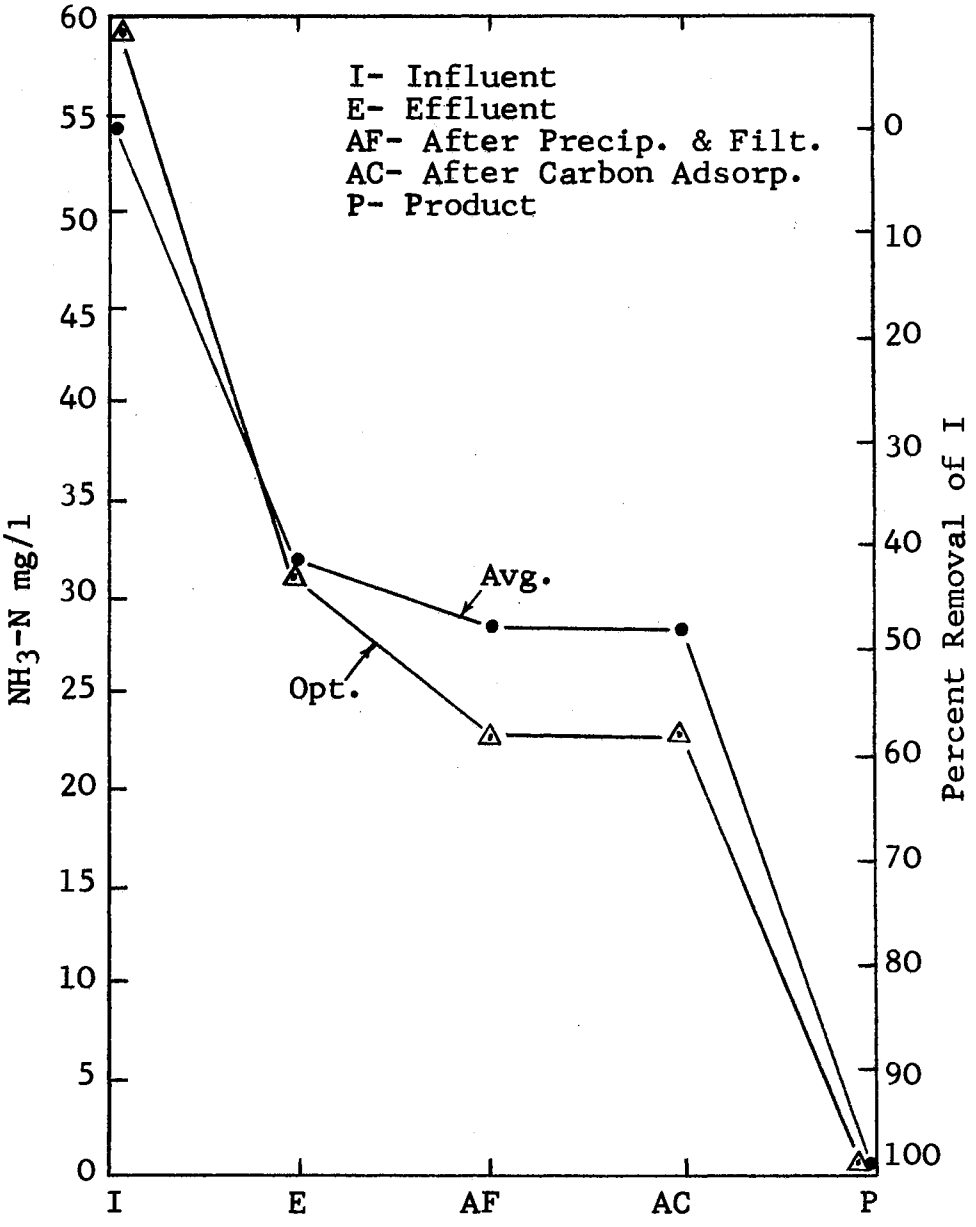

Figure 25. Ammonia-Nitrogen Removal By The Unit Processes.

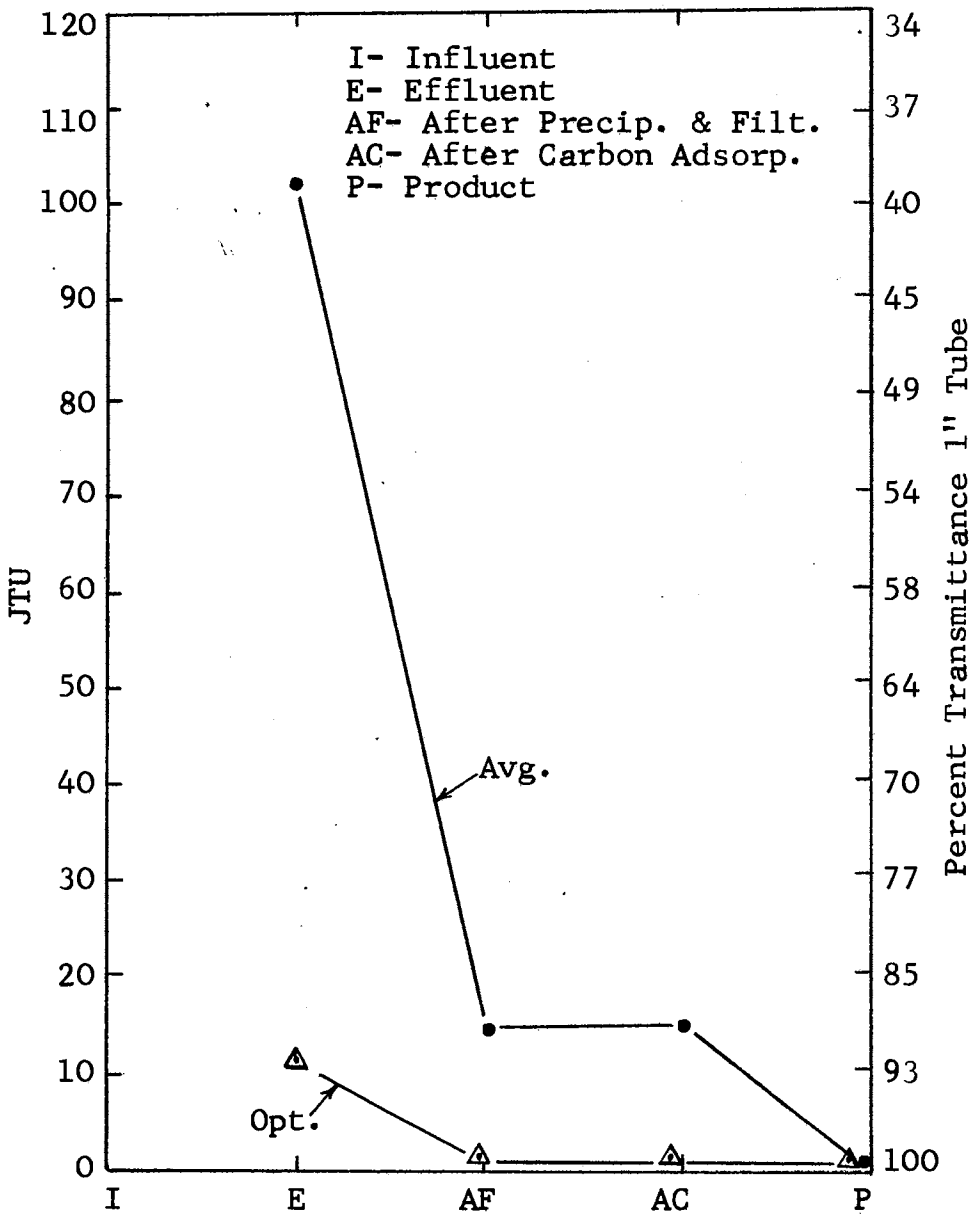

Figure 26. Turbidity and Transmittance Variation By The Unit Processes.

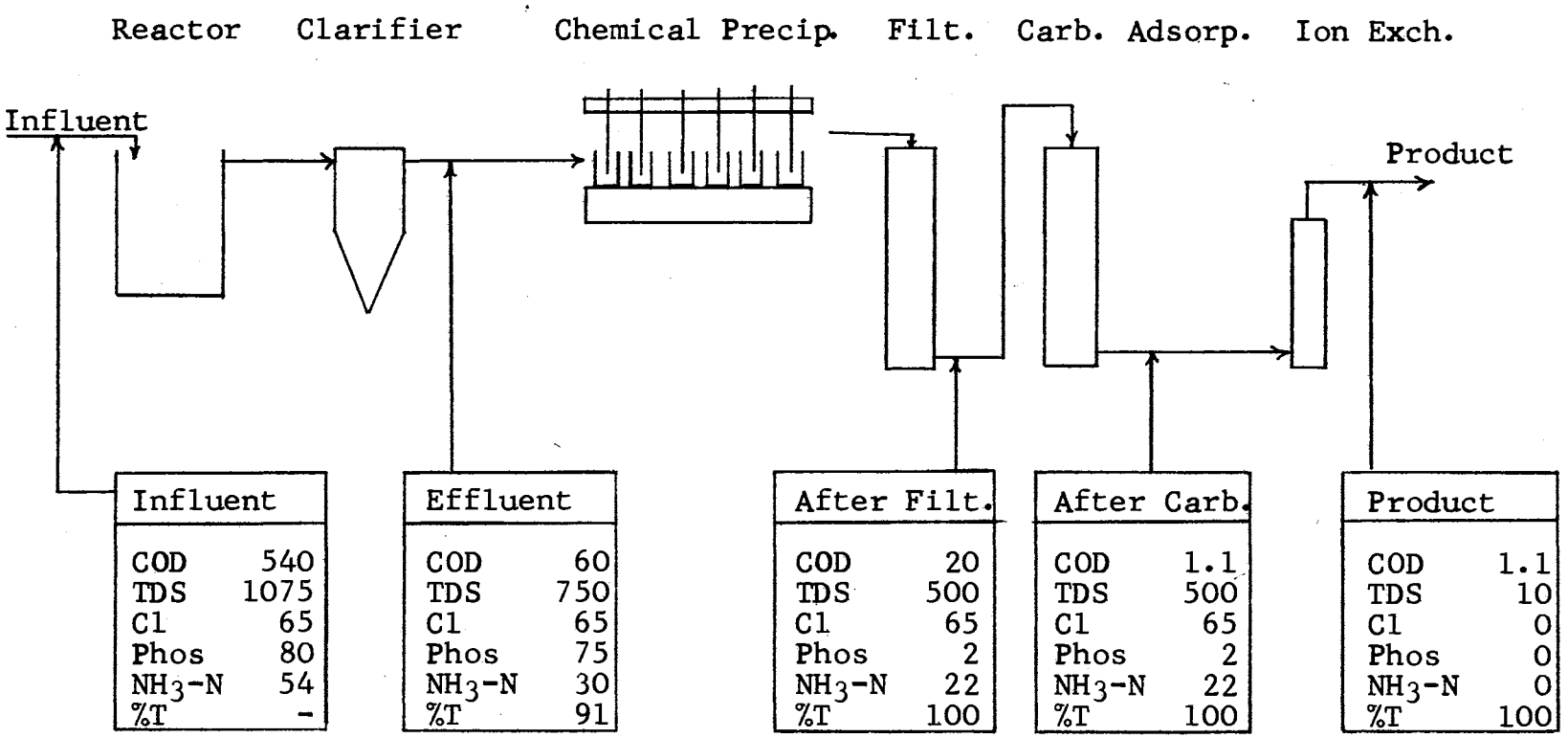

Figure 27. Diagrammatic Representation of Treatment Process Effectiveness.

CHAPTER V

DISCUSSION

It was hoped that by conducting this study an insight would be gained into the effect that continuous direct reuse of water would have on the quality of the water. The study was broken up into two phases, one that modeled reuse where only conventional wastewater treatment and conventional water treatment were utilized and the other where convention wastewater treatment and advanced wastewater treatment (or if one will, advanced water treatment) were utilized. Although a complete chemical analysis was not conducted on the water, enough chemical contaminants were analyzed to permit characterization of the water quality at any point.

From Phase One of the study it was learned that conventional water treatment, of even a good quality secondary effluent, was insufficient in producing potable water in a continuous closed system. It was seen that there was a residual COD in the water after treatment, that the TDS concentration increased substantially along with other compounds, and the clarity of the reused water decreased to the point where it would certainly be objectionable to people using it. This supports the findings of actual

field operation at Chanute, Kansas (14).

Referring to the clarity of the water, it can be said that of all the characteristics that were analyzed for, clarity, as expressed by turbidity or transmittance, is the most important factor as to whether reused wastewater would be accepted for domestic purposes. No person would willingly drink or use water that has visual color in it, but that same person would probably accept and use water with a TDS concentration of perhaps 2000 mg/l and hardly, if at all, notice anything.

For Phase Two of the study it was shown that no contaminants built up through continuous reuse and that potable water was produced consistently. Even though conventional wastewater treatment during Phase Two did not, on the average, produce a good quality secondary effluent, the AWT process was able to overcome these conditions. However, overcoming these conditions were at the expense of increased sludge production and the necessity of more frequent backwashing of the filter.

In general chemical precipitation and filtration were shown to be very effective unit processes for the removal of chemical contaminants. However, more effective removal could have been obtained with monitoring the pH and varying the lime dosage to obtain an optimum pH.

The activated carbon column was capable of removing between 67 and 95 percent of the COD that it received. This occurred even though the contact time was only 7.5

minutes whereas the recommended contact time was 40 to 50 minutes. From this fact and that information presented in the literature (34-37) it is believed that for a longer contact time close to 99 percent of the COD influent to the column could have been removed.

Cycle four of Phase Two showed that using the final clarifier as a chemical precipitation basin also, was ineffective when lime was used as the coagulant. Better results could likely have been obtained using a different coagulant dosed directly into the reactor as has been done in the field and shown to be relatively effective (30,31). However, in consideration of producing potable water it is felt that separate chemical precipitation should be adhered to.

Demineralization of the water was very effective and is probably the most important process if continuous reuse of the water is expected. Not all of the water would have to be demineralized to prevent a build up for any given contaminant. Since the ion exchange cartridges used for demineralization were of the mixed bed type they could not be regenerated. This would be impractical for field use and separate exchange resin beds (columns) would be required.

Another thing shown by the results of Phase Two is that water as clear as distilled water can be produced continuously. As discussed before this is probably the most important factor in the quality of the reused water, and it

is encouraging to know that the clarity of the water will not deteriorate through continuous reuse.

This study shows that wastewater can be reused for potable water directly and continuously when the proper treatment process is employed. Continuous reuse can be conducted without any deterioration of the water quality. A higher degree of control over the entire treatment process, than that normally practiced today, can and must be maintained such that potable water of any desired quality is produced consistently and economically.

CHAPTER VI

CONCLUSIONS

Conclusions made based on the results of this study are:

(1) The direct continuous reuse of wastewater as a source of potable water supply is feasible with present "state of the art" technology.

(2) Conventional water treatment alone is insufficient as a treatment process for reclaiming wastewater for domestic reuse purposes.

(3) Advanced wastewater treatment can overcome temporary upset conditions in the conventional wastewater treatment process, but AWT is only practical when a good quality secondary effluent is consistently produced.

(4) Chemical clarification should be monitored more closely and should be flexible to allow for changes in coagulant dosage.

(5) Demineralization is required for only a certain percentage of the water.

CHAPTER VII

SUGGESTIONS FOR FUTURE STUDY

Based on the findings of this study, the following suggestions are made for future study on water quality considerations of wastewater reuse as a source of potable water:

(1) A more detailed chemical analysis be conducted.

(2) A bacteriological-virological analysis be conducted along with the chemical analysis.

(3) Since make-up water will be required it should be determined whether it is better to mix the make-up water and the wastewater and then treat them together or to treat them separately and then combine.

(4) Investigate the use of polyelectrolytes (coagulant aids) for obtaining better chemical precipitation.

(5) Determine the optimum percentage of water to demineralize in terms of economics and water quality.

(6) Since the reused water must be "safe", aesthetically pleasing, and produced continuously the treatment process will have to be somewhat overdesigned. How and how much the process will require in overdesign needs determination.

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VITA 2

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